VOL. XXXVI. No. 7. JULY, 1958

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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE INSTITUTION OF STRUCTURAL ENGINEERS

Some Structures Involving Unusual Design Problems
by Clifford E. Saunders (Delegate Member of Council)

Some Notes on the Use of High Preload Bolts in the United Kingdom
Discussion on the Paper by F. M. Easton (Associate-Member), E. M. Lewis and D. T. Wright

The Problem of Mechanical Handling in Building Operations
Discussion on the Paper by J. F. Eden and D. Bishop

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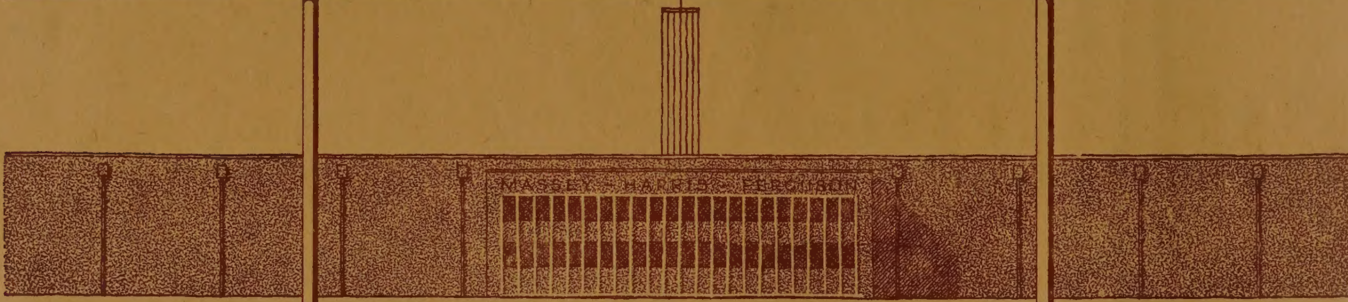
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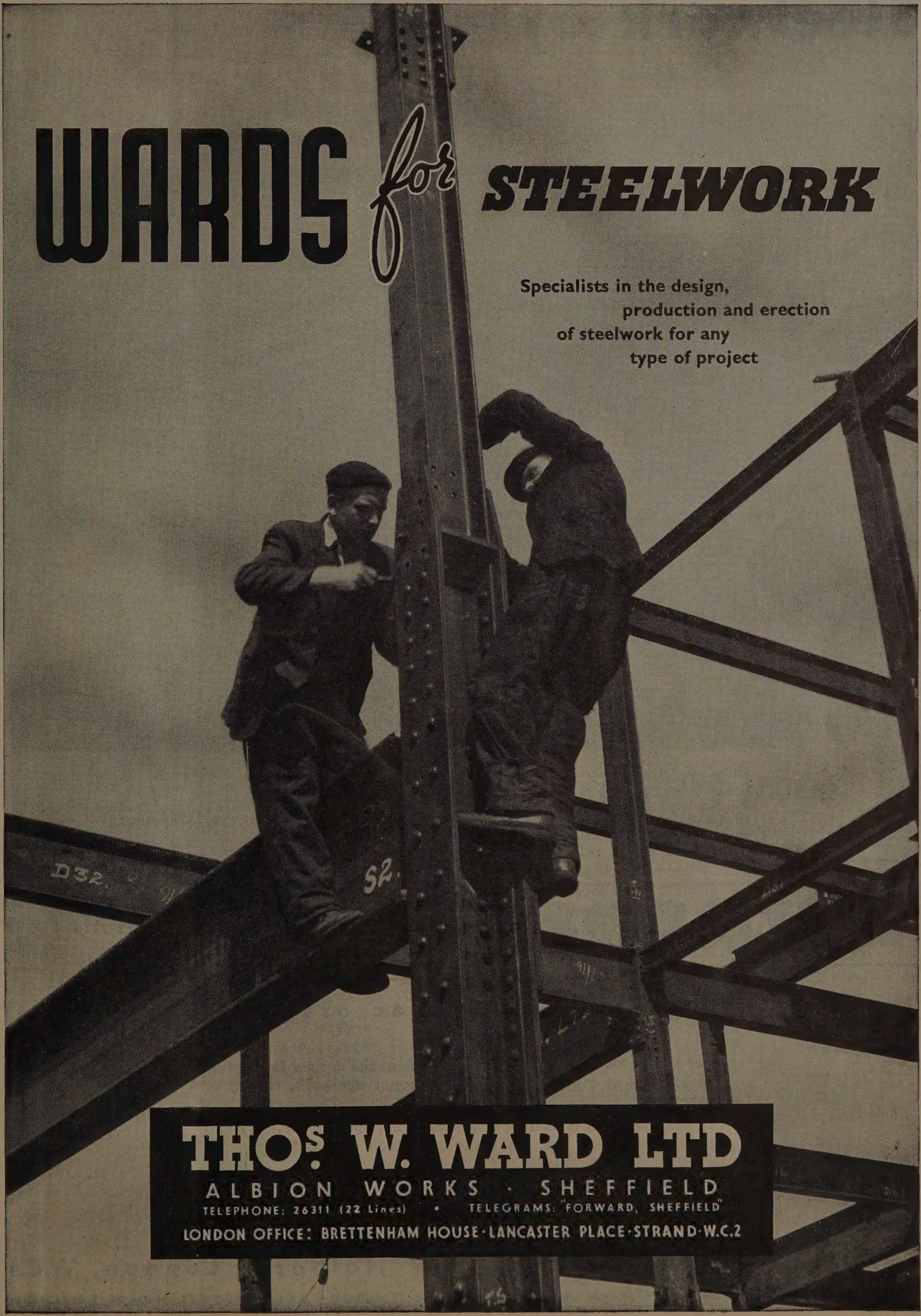
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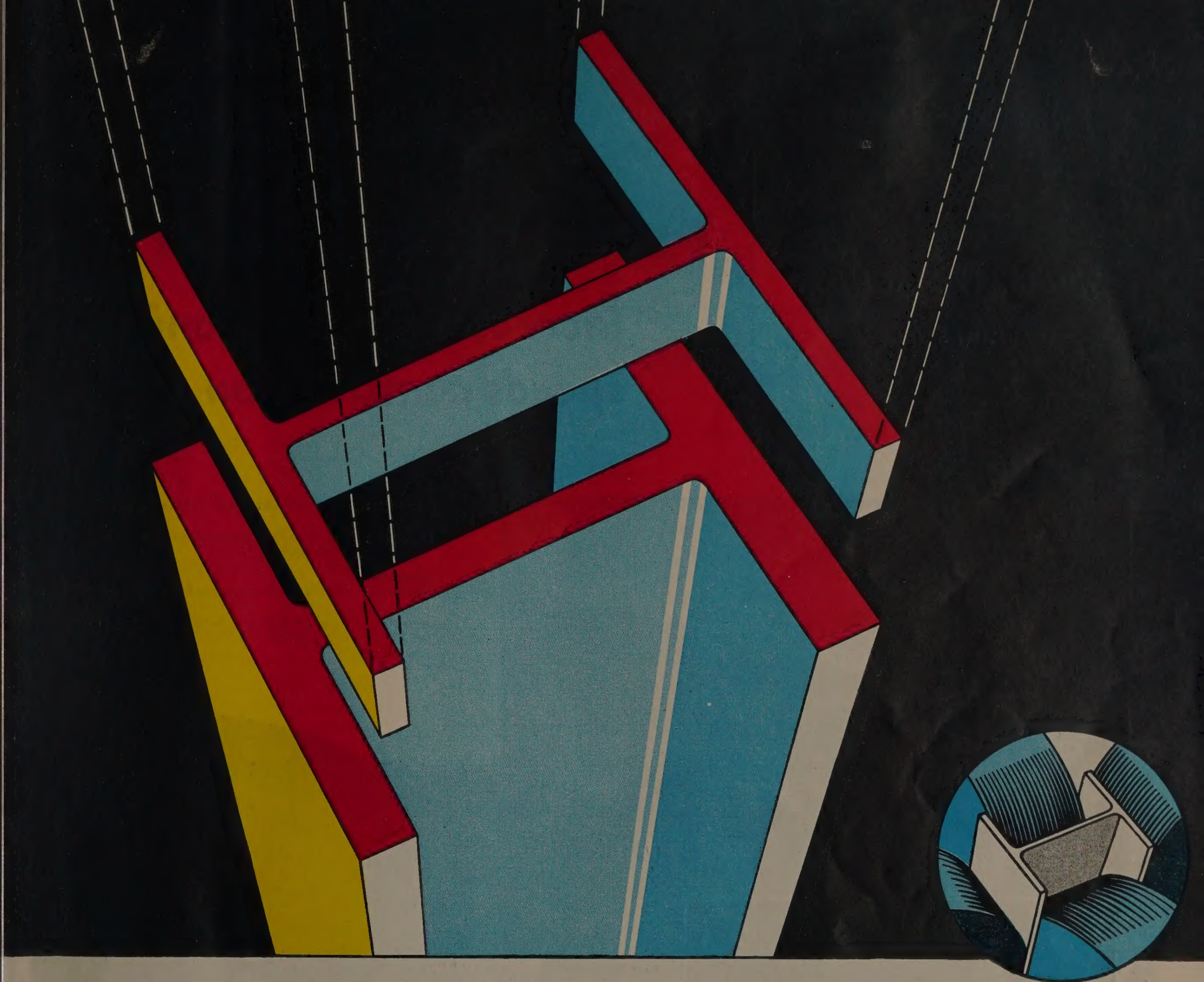

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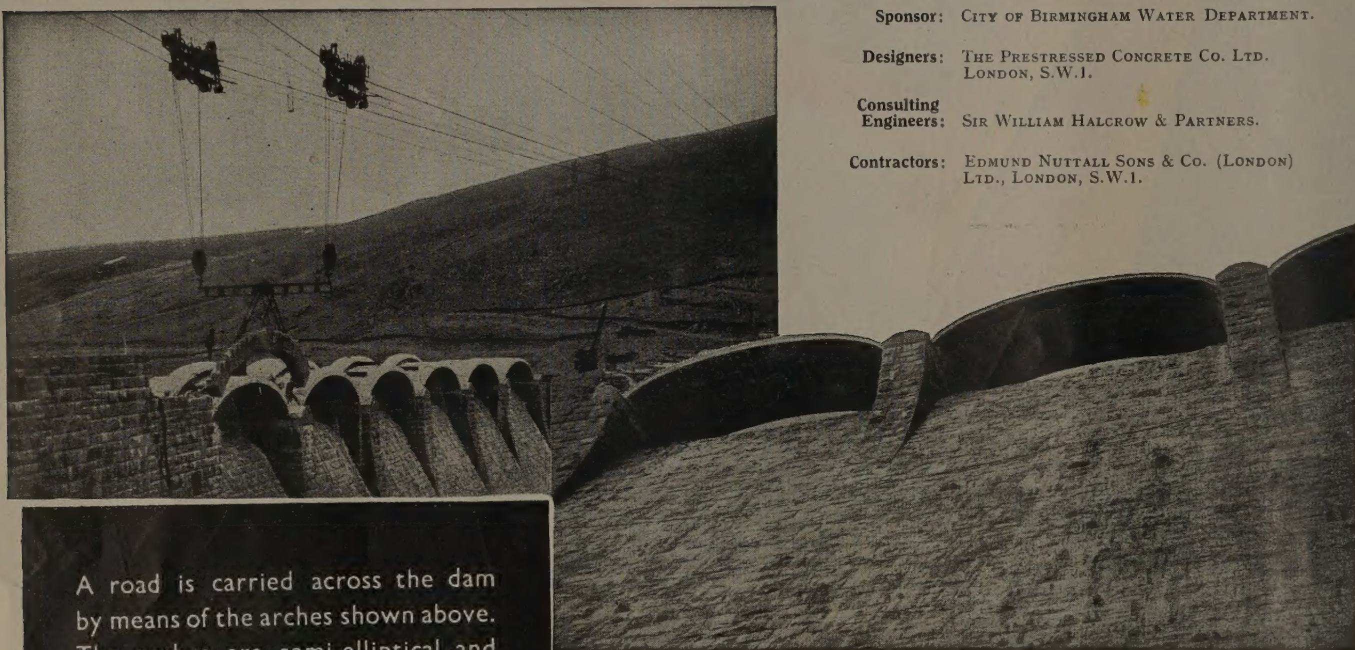

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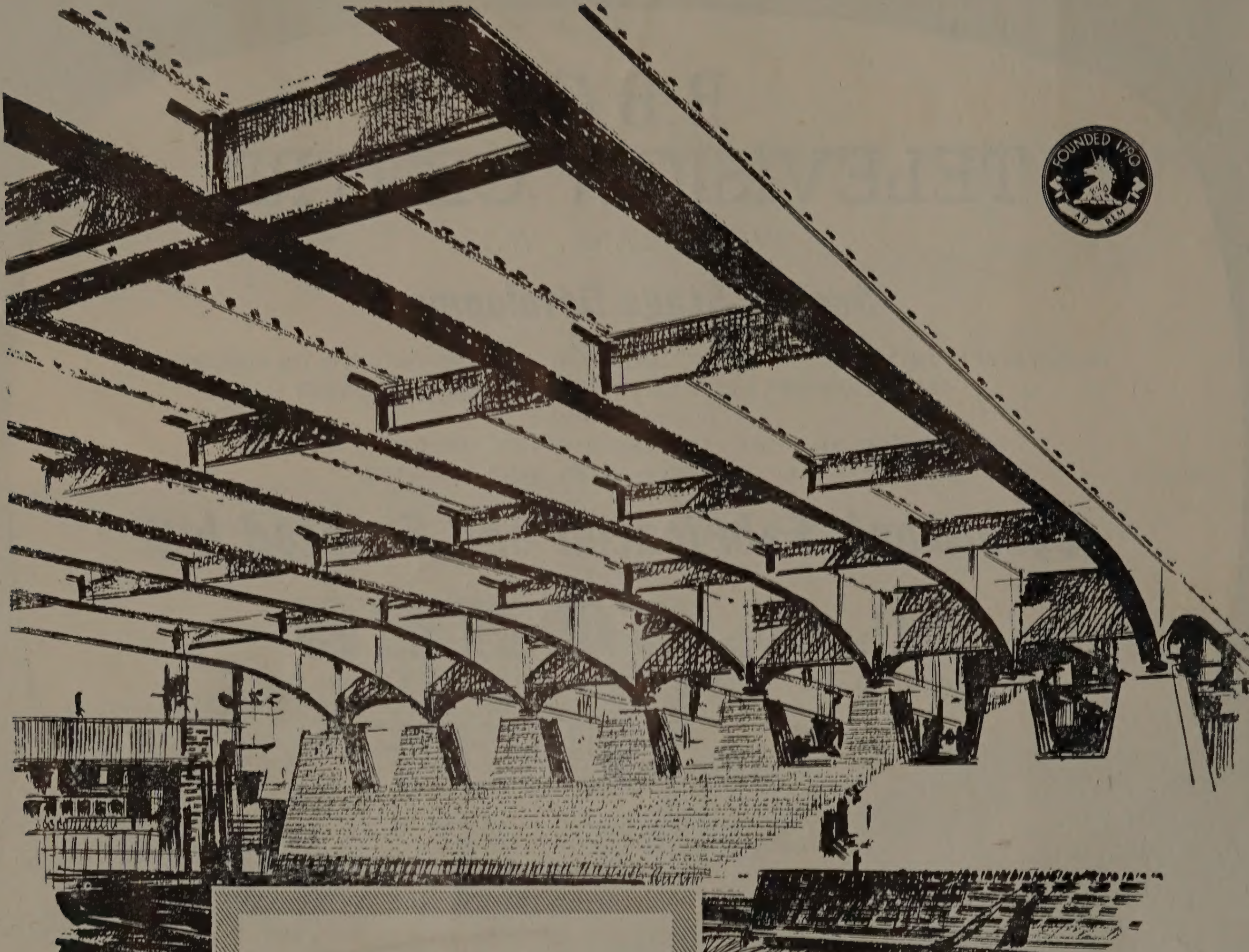

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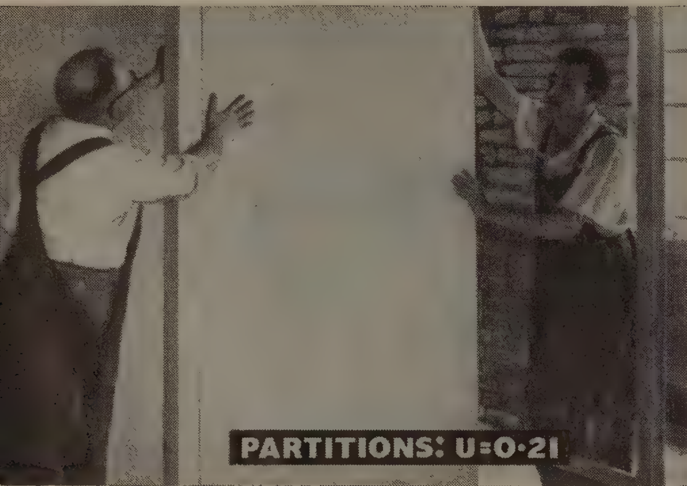

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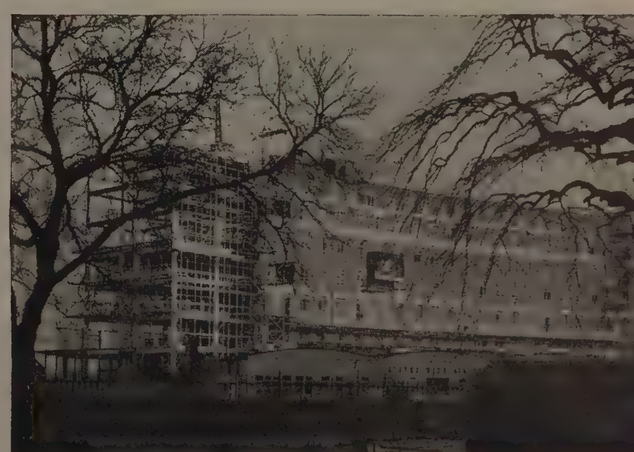

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Consulting Engineers: Wheeler & Jupp, M/M.I.C.E., M/M.I.Str.E.
Hurst, Peirce & Malcolm, M/M.I.C.E., M/M.I.Str.E.
Main Contractors: Humphreys Limited.

Above right: new building for 'The Financial Times'

Architects: Sir Albert Richardson, P.P.R.A. and E. A. S. Houfe, Esq., F.R.I.B.A.
Consulting Engineer: W. A. Mitchell Esq., M.I.Str.E., M.I.W.
Contractors: F. G. Minter Ltd.

At right: new offices for the Bank of England

Architects: Victor Heal & Smith, F/F.R.I.B.A.
Consulting Engineers: Hurst, Peirce & Malcolm, M/M.I.C.E., M/M.I.Str.E.
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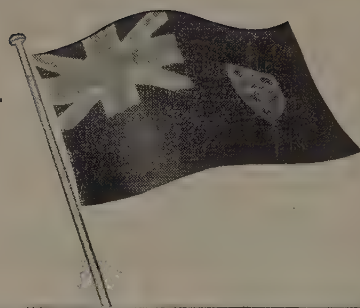
At your Service almost  *everywhere*

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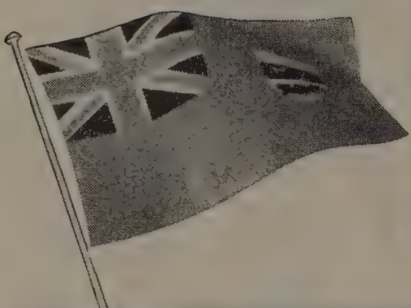

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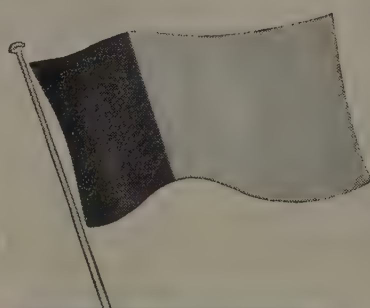

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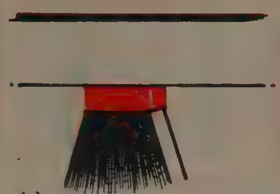
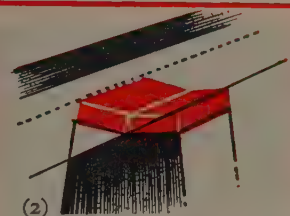
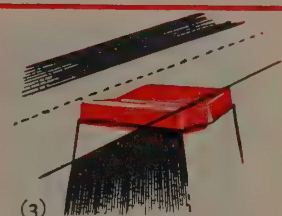


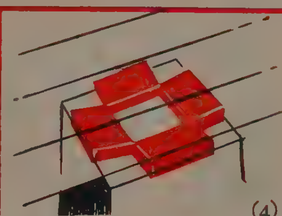
(1) One mounting placed flat replaces two sets of roller bearings, one on top of the other, and a rocker bearing.



(2) Two mountings placed as a V in the line of the bridge span replace one rocker and one roller bearing and provide lateral stiffness.



(3) Two mountings placed as a V across the line of the bridge span replace a straight rocker and provide longitudinal stiffness.



(4) A combination of (2) and (3) in the form of a square replaces a spherical bearing.

The end of the Pelham Bridge where it joins the approach. The rubber bridge mountings can be seen at the top of each of the eight columns.

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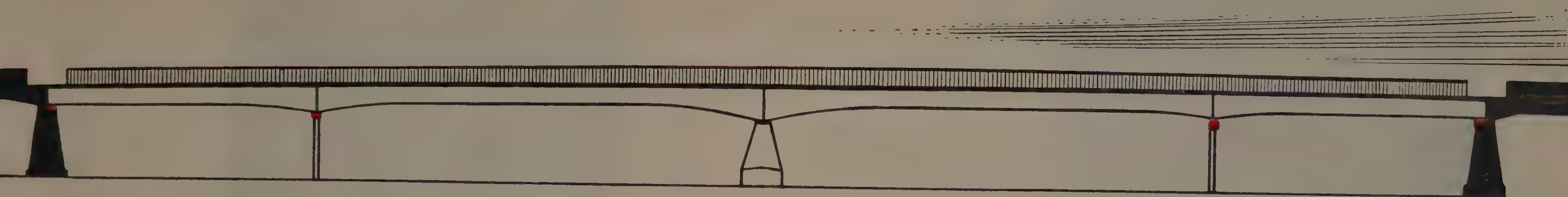

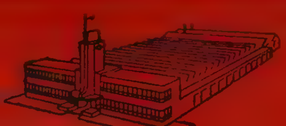

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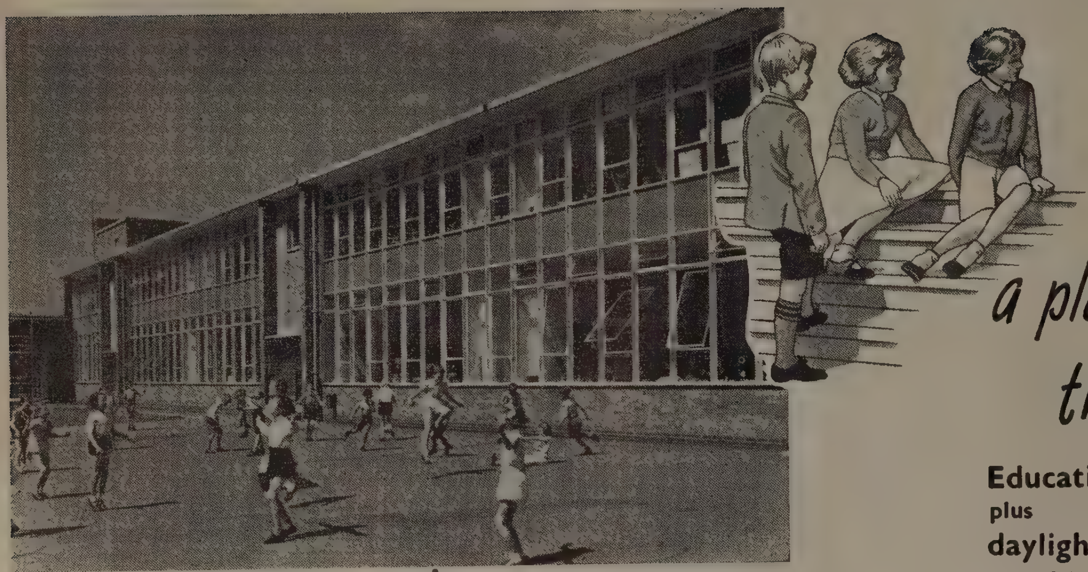

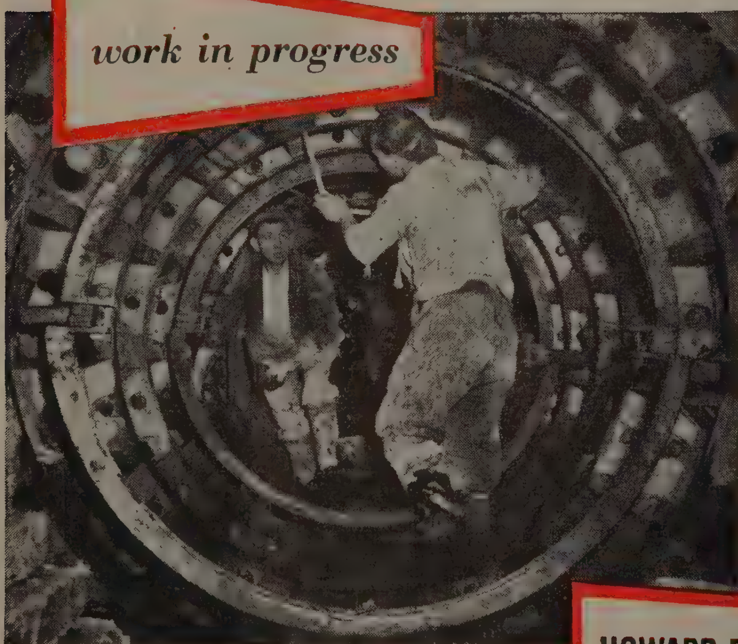

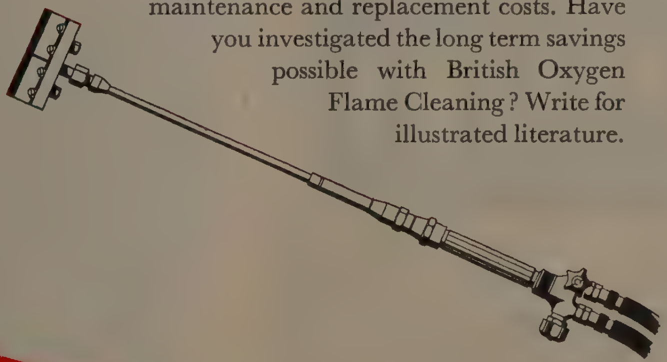

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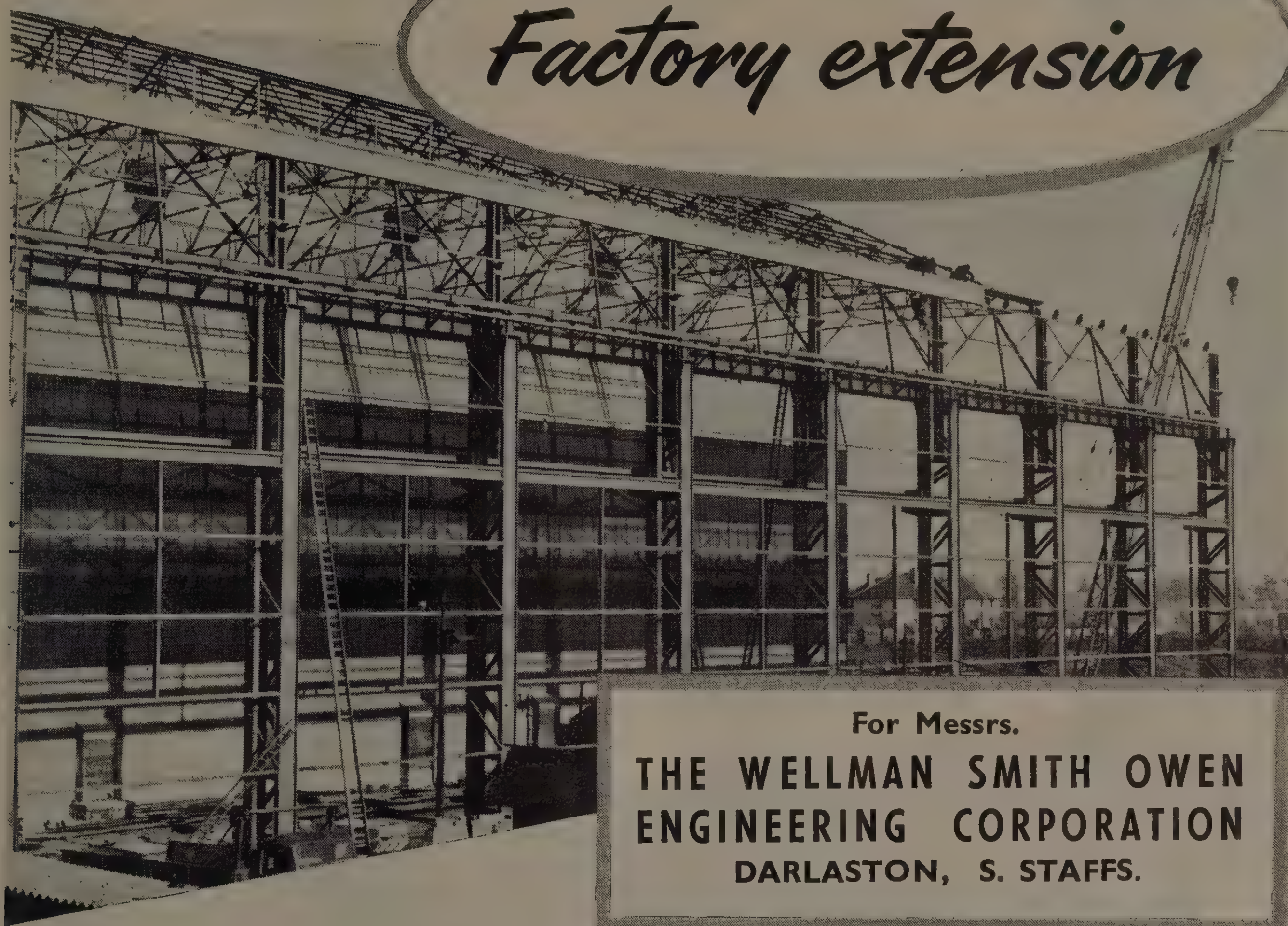

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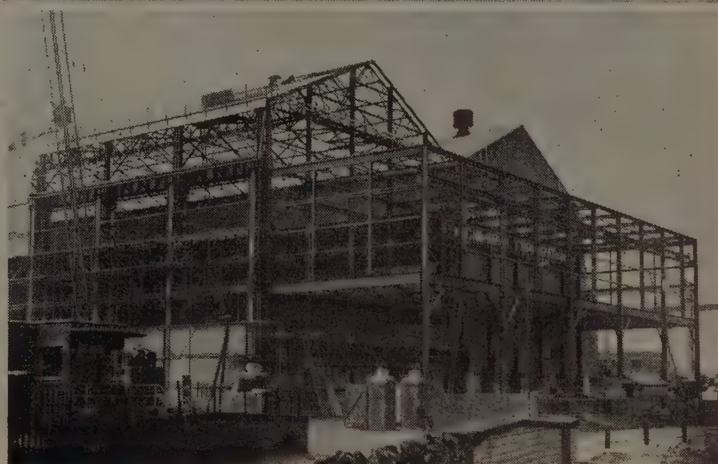

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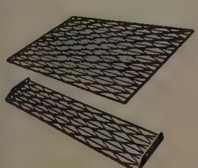

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VOL. XXXVI. No. 7. JULY, 1958

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Some Structures Involving Unusual Design Problems*

By Clifford E. Saunders, M.I.Struct.E., A.M.Inst.W.

Introduction

It must be true to say that every trade and profession offers some advantages and disadvantages to its members, but undoubtedly one of the attractions of structural engineering is the immense degree of variety that arises in its work and problems.

Every engineer dreams about the large, square, repetitive and straightforward jobs with their hundreds or thousands of tons of steel, but it often happens that smaller projects involving unusual design problems or construction difficulties provide at least as much satisfaction to the engineer from the standpoint of his professional, if not his financial, interest. It is with a few jobs of this type that the author will deal in this paper.

Cable Loading Gantry, Erith, Kent

The loading structure at Erith, Kent, for transferring submarine cable from factory to cable-laying ship was specially erected in the first instance for the recently completed Transatlantic Cable, and some may have seen the gantry featured in news-reels or television programmes which from time to time have illustrated various phases in the manufacture, loading, and laying of the cable.

As many descriptions of submarine cables in general, and the Transatlantic Cable in particular, have been published there is little point in describing it in detail here, but, briefly, in principle it comprises a copper wire with thin strands of copper tape wound around, covered with polythene insulation, then more copper tape and sheathing, jute, steel armour wire to withstand tension, more jute, and pitch, giving an outside diameter of about $1\frac{1}{4}$ in. The detailed construction of any particular stretch of cable is, of course, dependent upon its particular requirements and location; for example, near land or in shallow water cable is usually more heavily armoured on account of the greater risk of damage through movement on the sea bed.

At intervals throughout the length of cable, and in the case of the Transatlantic Cable about every 40 miles, is a repeater or booster unit containing very costly electronic equipment, each repeater being purpose-designed and made, and therefore demanding the most careful and gentle handling. While the cable itself is fairly flexible and can be curved to a radius of a few feet, the maintaining of straightness was most strictly enforced for a stretch of some 30 feet containing each repeater, and the design of the loading structure very largely hinged around this requirement.

* *Presented to the Western Counties Branch of the Institution of Structural Engineers at a combined meeting with the Institution of Civil Engineers on December 7th, 1956, to the South Western Counties Branch on 29th November, 1957, and to the Wales and Monmouthshire Branch on 19th March, 1958. Western Counties Branch Prize-winning paper, 1956-7.*

The cable was made in a factory constructed adjacent to the river Thames, and was fed by means of a gantry to a tank house where it was stored in large tanks (Fig. 1). In the tank house the repeaters were spliced into the cable, each repeater linking lengths of cable stored in adjacent tanks.

In the river a channel was specially dredged to enable H.M.T.S. "Monarch", the largest cable-laying ship in the world, to approach to within 650 feet of the tank house. Three dolphins were constructed in the river for mooring the ship.

The problem was to convey the cable, with repeaters already inserted, from the tank house to the ship, bearing in mind the following requirements:

- Support for the cable had to be provided at least every 6 ft.;
- Sharp curvatures or radii were not permissible on account of the repeaters, for which straightness was stipulated at any rate until loaded in the ship;
- Three lines of cable might be required to pass simultaneously from the tank house to the ship;
- The first 175 feet had to pass over a wharf, the remaining 460 feet to 610 feet being over water (this dimension being dependent upon the position cable enters the ship);
- Trestles or towers were undesirable on the wharf, firstly because of obstruction and secondly because of the expense of piling foundations;
- Only one intermediate tower could be permitted in the water on account of obstruction to river traffic constantly using adjacent wharves;
- While the part of the structure over the wharf could be permanent, that over the river had to be demountable in the interests of river traffic, and a minimum clearance of 42 feet above high spring tides was stipulated for the span between the towers;
- Maximum tidal range, large and small ships both light and laden, and various points of entry into the ships, all had to be catered for, the highest extreme being the largest ship (H.M.T.S. "Monarch") light, at high spring tides, and the lowest a small ship, laden, at low spring tides, (Fig. 2);
- The various points of entry of the cable into the ship introduced a plan range embracing an angle of 50°, taken from the river tower, i.e., 25° to either side of the axis of the gantry.

In view of the foregoing requirements the structure was designed in three spans, viz.:

- A permanent bridge 175 feet long and 38 feet high above the wharf, spanning from the tank house to a tower at the edge of the wharf and known as the wharf tower;
- A wire rope span of 260 feet spanning from the wharf tower to the river tower;

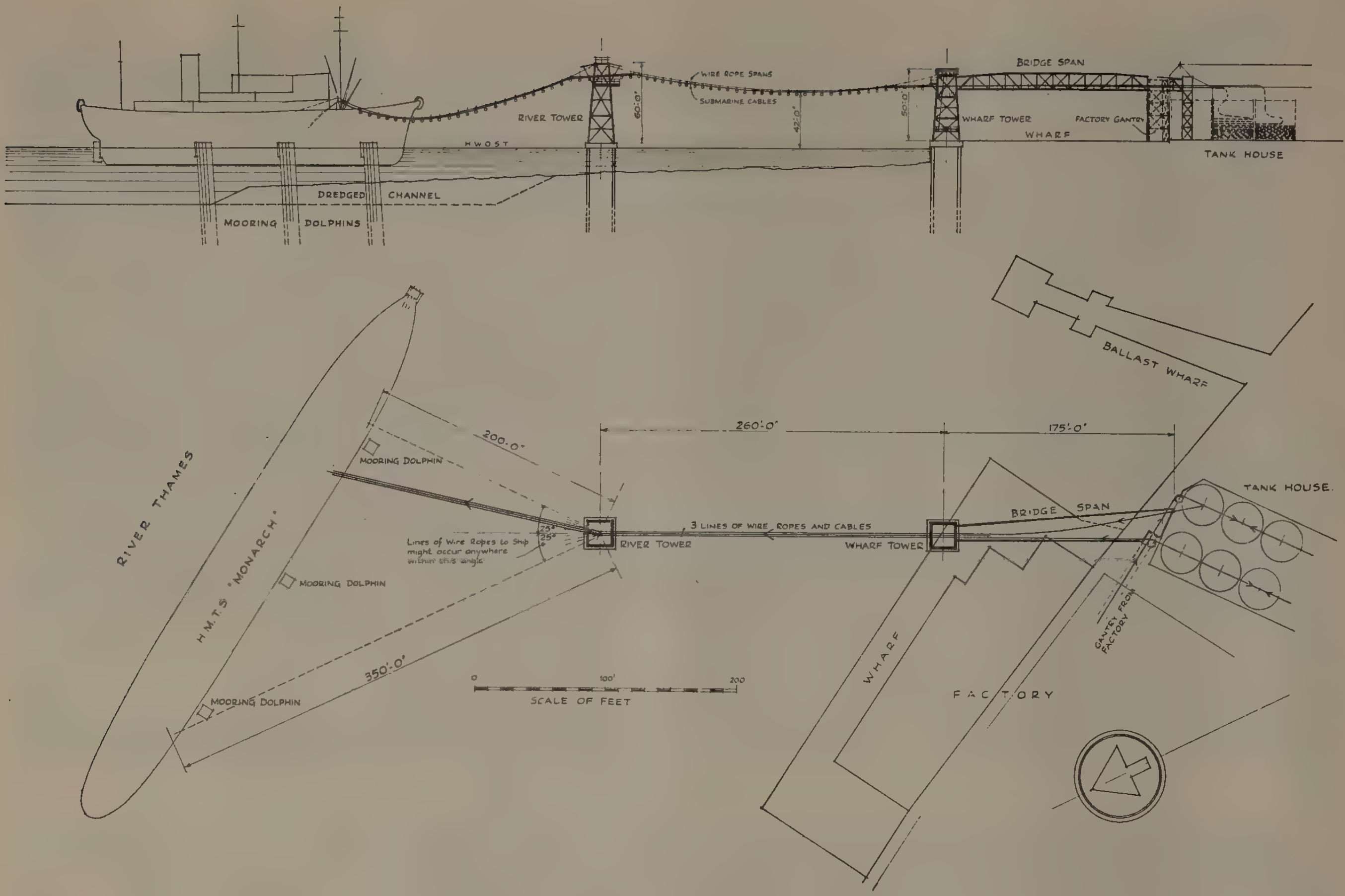


Fig. 1—Site Layout Erith

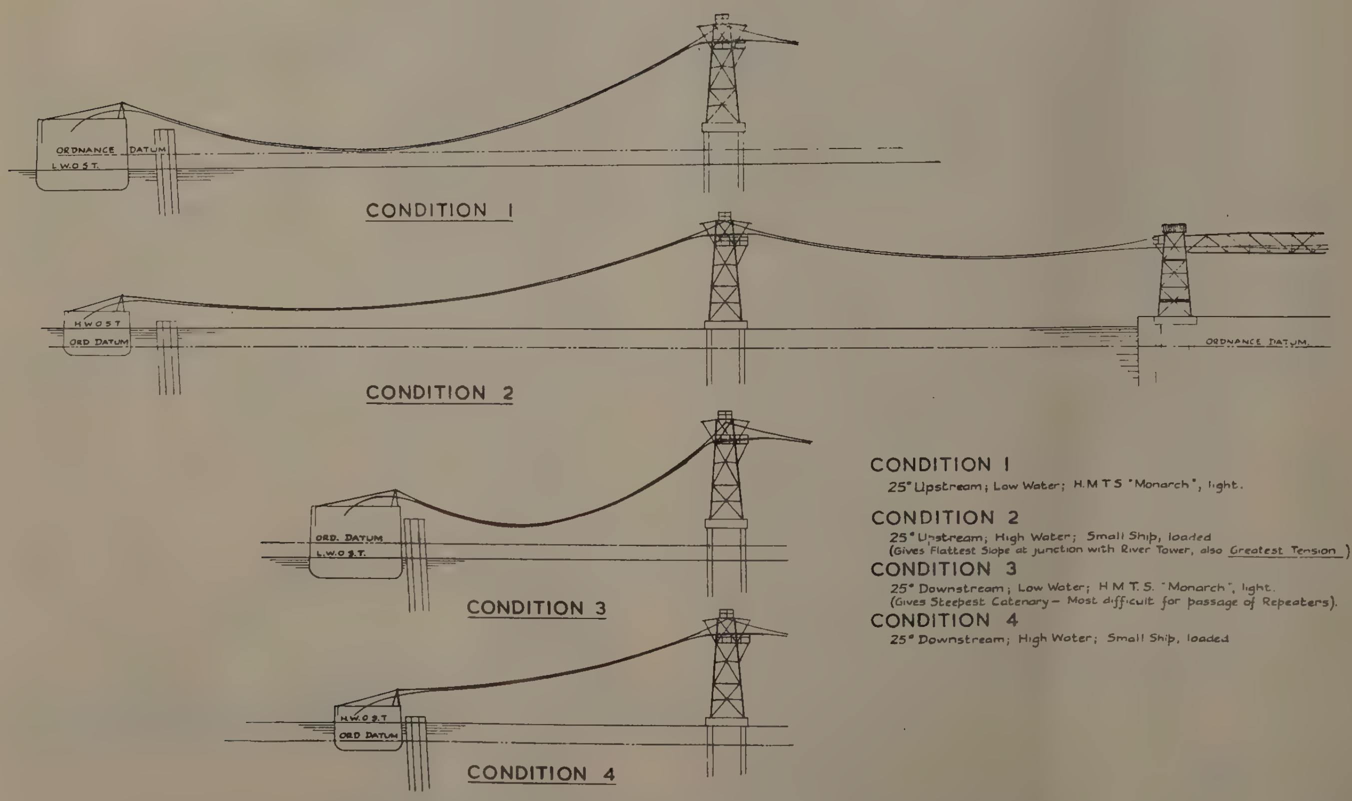


Fig. 2.—Various Conditions of Cable Loading and Tides

A second wire rope span varying from 200 feet to 350 feet spanning from the river tower to the ship (the dimension being dependent upon the point of entry of cable into the ship).

The two towers are each supported on a dolphin, constructed of steel piles about 80 feet long surmounted by a reinforced concrete deck. (The two tower dolphins and three mooring dolphins were constructed under the supervision of Sir William Halcrow & Partners).

The bridge spans 145 feet on one side and 175 feet on the other, and tapers from 11 ft. 6 in. wide at the wharf tower end to 28 feet wide at the tank house end. This irregular plan was necessary to enable cable runs to be taken from either of the two lines of storage tanks without sudden change in line, (Fig. 3).

bracing and the panel points of the main girders. The longer compression members are generally of cruciform section, and the three very long cross braces at the wide end of the bridge are of fish-belly type, comprising tees top and bottom with flat lattice bars welded on. This type was adopted for lightness of appearance because the great length (40 feet) necessitated a fair depth to reduce deflection, and had the braces been of parallel or solid web section they would have appeared very heavy in comparison with the booms of the main girders.

Goal-post type frames are fixed across the bridge at every six feet throughout its length. Sheaves can be attached to the posts of these frames or slung from the cross beams at the required positions and levels to support the cable as it passes across the bridge.

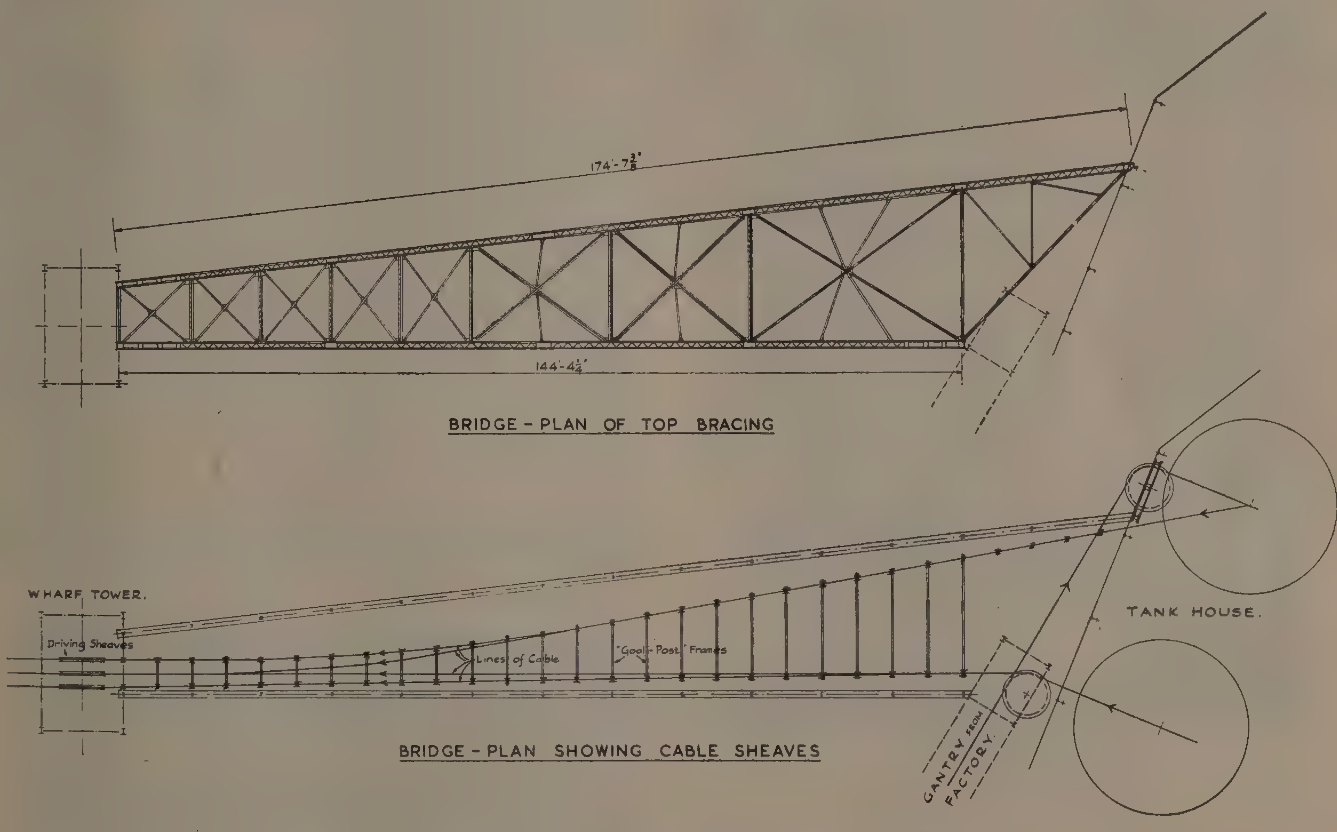


Fig. 3.—Plans of Bridge Top Bracing and Cable Runs

The main girders are of riveted and welded construction, the members being riveted together with gusset plates, but batten plates and lattice bars being welded to the members after riveting up of the girder sections. The girders were shop riveted into sections 36 feet long with a maximum depth of 12 ft. 9 in., and the sections were site assembled with turned and fitted bolts. The floor is of chequer plate with orthodox wind bracing below.

Achieving a clean and light appearance in the top wind bracing presented a problem on account of the width and the tapering plan of the bridge, but this was overcome by cross bracing five bays in the usual manner, then cross bracing two pairs of bays, then one set of three bays, lateral ties being inserted in the two latter cases between the crossing points of the

At the tank house end the bridge is supported on one side by a trestle in the line of the tank house gable, and on the other side by the tower of the existing gantry passing cable from factory to tank house (see also Fig. 10). This end of the bridge is fixed.

The trestle serves two purposes—supporting one corner of the bridge, and housing at the top a bank of three 7-foot diameter sheaves around which cable passes en route from the factory to the tank house. It comprises two stanchions and bracings, forming a vertical lattice girder, shop riveted up in two panels, site assembled with turned and fitted bolts, and is designed to resist the horizontal forces from wind pressure on the bridge and from the passage of cable around the sheaves. The trestle is supported on a foundation of piles, outside the tank house, and a

cantilevering pile cap. At the top the trestle is tied laterally to the tank structure inside the tank house.

The supporting of the other corner of the bridge off the stanchion of the existing tower proved rather an embarrassment. This tower is supported on piled foundations, but while the piles were able to withstand the additional load imposed by the bridge main girder reaction, the additional load was too great for the stanchion, hence it was necessary either to strengthen the stanchion or to introduce another new stanchion by the side of it. Compounding the stanchion would have introduced complications as the tower bracings would have required modification, and this would have involved their temporary removal which, in turn, would have put the tower out of commission—an inconvenience that could not be tolerated as cable was passing day and night from the factory to the tank house. On the other hand, the insertion of an extra stanchion close to the existing would have introduced eccentric loading on the piles which would have overstressed them, and it was desired, as far as possible, to avoid driving new piles.

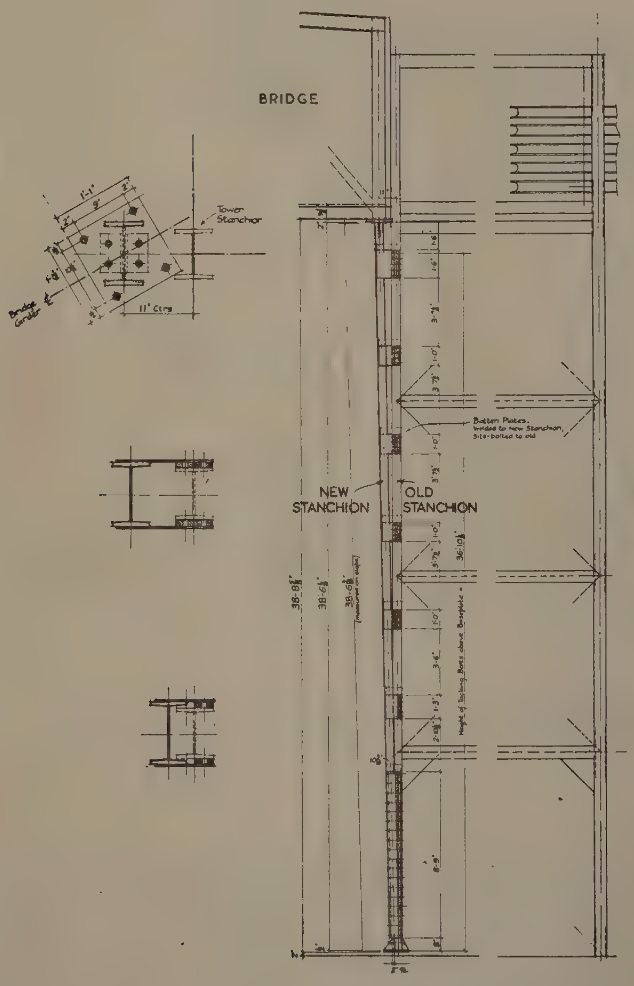


Fig. 4.—Strengthening Stanchion of Original Tower

The difficulty was overcome by inserting an additional stanchion leg, battened to the existing (Fig. 4) but raking the new leg to bring the combined load on to a modified base, the centre of which practically coincided with the centre of the original, hence the eccentricity of the load on the piles was reduced to negligible proportions. To bring the centreline of the new leg sufficiently close to that of the old stanchion it was necessary to remove a tapering section from each flange of the old stanchion, but the operation was completed without causing interference to the process.

The bridge main girder was hoisted into position almost immediately, and the only difficulty occurred when somebody not acquainted with the matter spread a full scale alarm to the effect that we were pulling the existing gantry over, and that it had already moved several inches out of upright !

The other end of the bridge is supported by rocker bearings off a box plate girder in the wharf tower. These firstly cater for expansion and contraction, and secondly ensure that horizontal forces imposed on the wharf tower by the first wire rope span are not transferred through the bridge to the tank house structure, but instead are resisted solely by the wharf tower dolphin, which is free to deflect horizontally, inducing bending in the piles.

As mentioned previously the catenary of the wire rope span between the towers was dictated by the minimum clearance above high spring tides, while the catenary for the span to the ship was governed by tidal range and size and state of loading of the ship. In addition it was stipulated that the submarine cable should not touch river water en route to the ship. The most extreme of these conditions resulted in rather flat catenaries, which give a maximum calculated tension in each wire rope of 33 tons, i.e., 99 tons for the three wire ropes, which might be loaded simultaneously. It was for these wire rope tensions, in combination with side pull and wind, that the towers were designed, although some relaxation to the catenary requirements was made subsequently for reasons connected with the dolphins.

The anchorage of the wire ropes to the ships required careful consideration, to avoid excessive tension being applied through the wire ropes to the river tower in the event of movement of the ships from tidal or navigational conditions. A length of nylon rope was therefore incorporated at the end of the wire ropes for the sake of its high elasticity and this was calibrated to provide a visual warning against excessive tension. The anchorage of wire ropes to the towers will be described later.

Sheaves are suspended at 6 feet centres off the wire ropes to support the cable as it passes along. At the ends of the wire rope spans adjacent to the towers secondary ropes are slung from the main to carry the sheaves, in order to maintain a straighter course for the cable and to assist in the passage of the repeaters.

The river tower is about 60 feet high from the top of the dolphin × 20 feet square in plan at the base. The most adverse set of loading conditions for its design arises from the span on one side having three wire ropes fully loaded while those on the other side remain unloaded (it being unlikely that the span on one side would be loaded while the other side is unrigged) and from the side pull when the point of entry into the ship is at the maximum angle upstream from the axis of the gantry, coupled, of course, with the wind forces liable to blow from any direction in this position of great exposure. It is constructed of 12 in. × 8 in. R.S. Joist stanchions and double channel bracings, torsion bracing also being incorporated. The stanchion bases are of welded construction, each stanchion being attached to the dolphin by means of four 2½ in. diameter holding-down bolts 4 ft. 4½ in. long, with channel anchors. The sixteen holding-down bolts for the tower were set solidly into the dolphin

concrete and were positioned by means of a steel jig. The gussets connecting the bracings to the stanchions were of tee type, fabricated by welding two plates together with small stiffener fins.

At the top of the tower is a horizontal lattice girder to which the six wire ropes are attached by means of shackles. The members of the lattice girder are turned-bolted together.

Above the working platform there are outriggers on both sides of the tower, these supporting tackle used during the passage of repeaters.

The wharf tower is similar to the river tower in its main construction, and also measures 20 feet square at the base, but is 50 feet high. The main difference is the method of anchoring the wire ropes since a means of adjustment to the catenary had to be provided. This was achieved by passing the wire ropes over sheaves incorporated in the top horizontal lattice girder (see also Fig. 8), the ropes continuing down the tower and attaching to chains, the links of which connect to anchors in a rigging platform at first floor level.

Another difference between the two towers is that the top floor of the wharf tower houses a machine with 7 foot diameter driving sheaves to assist in pulling the cable from the tank house to the ship.

The laying of submarine cables across the Atlantic Ocean is only possible in certain summer months when reasonably calm weather can be expected, consequently a dead-line date was laid down for the commencement of loading cable into the ship, and was most strictly enforced, particularly on account of the international importance of the project. This loading date was actually less than three months after functional design requirements were finalized, and the only way in which completion in time was possible was by fully broken-down detailing in the design office of every member and plate, almost every one of which throughout the job was different. This method enabled the steelwork contractor to commence fabrication five days after the design requirements were agreed, to commence erection a week later, and to maintain a flow of material to the site. All material was despatched piece-small with the exception of the shop-riveted panels of the bridge main girders and the trestle at the tank house end.

Fig. 5.—River Tower, Early Stages of Erection

The river tower was handled first. Fig. 5 shows it in course of erection, the method being fairly orthodox except that it had to be carried out from a barge. The wharf tower was erected next by means of a mobile crane on the wharf. Fig. 6 shows the first stages of this—the river tower, in a fairly advanced stage, and the mooring dolphins, can be seen behind.

Fig. 6.—Wharf Tower, Commencement of Erection

It was intended to erect the bridge by means of a 120 foot jib scotch derrick, to support which special dolphins were constructed adjacent to the wharf. However, the erection plan had to be modified as a result of the most disastrous collapse of the derrick while undergoing its loading test, fortunately without loss of life. It was quite impossible for another crane to be obtained and installed in time to suit the programme, so in consequence the bridge main girders were erected in sections by means of the mobile crane, temporary trestles being utilized for intermediate support, (Fig. 7). This illustration also shows the original gantry from factory to tank house.

Fig. 8 illustrates the wharf tower completed and rigged ; Fig. 9 is a view taken on the bridge during erection of the " goal-post " frames for the sheaves, illustrating in particular the fish-belly latticed members of the top wind bracing ; and in Fig. 10, taken under the bridge, can be seen the tank house, the trestle

Fig. 7.—Bridge Erection with Temporary Trestles

Fig. 8.—Wharf Tower, Structure Completed and Rigged

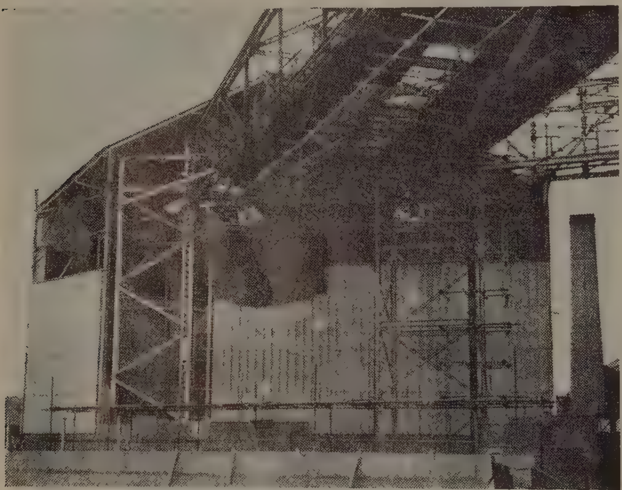


Fig. 10.—View under Bridge, Showing Tank House, Trestle and Original Gantry Tower

Fig. 9.—View Taken on Bridge

By courtesy of "The Times"

Fig. 11.—River Tower and H.M.T.S. "Monarch" Loading

supporting the bridge, and the original gantry from the tank house and its strengthened stanchion. Figs. 11 and 12 show the gantry working, with H.M.T.S. "Monarch" loading Transatlantic Cable, and Fig. 13 is another picture of the completed installation.

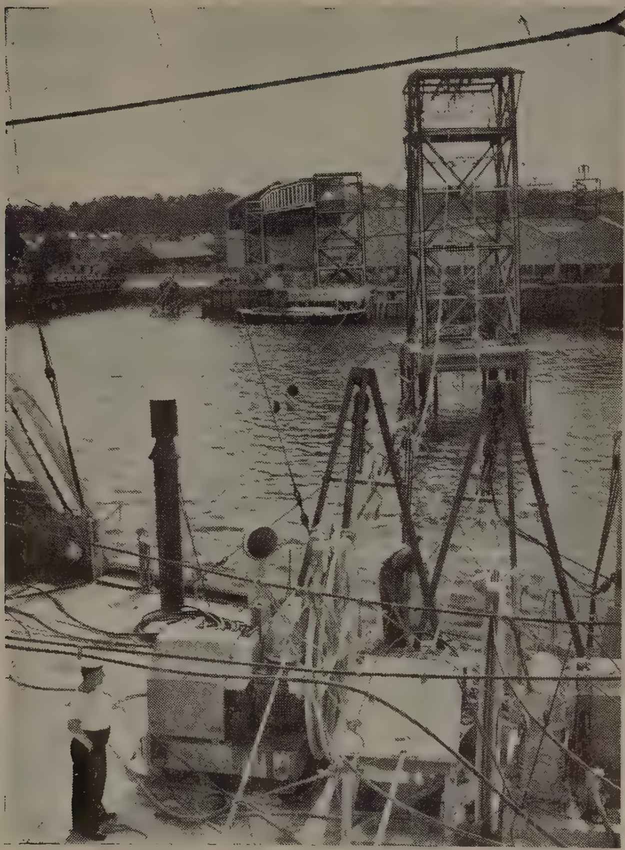
Notwithstanding the difficult circumstances that were encountered the installation was in working order on time (although without ladders and handrails, etc.), and the "Monarch" sailed five days early to start laying the cable.

Cable Loading Structure, Southampton

Another cable loading structure at Southampton is based on similar functional requirements but as the site conditions are completely different, the structure has little in common with that at Erith. In this case the 500 feet which the cable has to traverse between the tank house and the ship is entirely over land, but the structure crosses nine railway tracks and two roads. The main limitations due to site peculiarities were observing the requisite clearance between the railway

By courtesy of "The Times"

Fig. 12.—H.M.T.S. "Monarch" Loading, from the top of the Wharf Tower



By courtesy of "The Times"

Fig. 13.—Installation Viewed from H.M.T.S. "Monarch"

tracks and stanchions, and a minimum distance of 60 feet from the edge of the quay to the nearest piles, but within these limitations it was possible to adopt fairly short spans.

The design requirements for this installation were :

- Support for the cable had to be provided at least every ten feet ;
- Sharp curvatures or radii were not permissible ;
- Four lines of cable might be required to pass simultaneously from the tank house to the ship ;
- While the structure could, in the main, be permanent, a short section at the factory end and the 60 feet over the wharf had to be demountable ;
- A walking-way was required along the permanent structure ;
- Maximum tidal range, variety of ships, and states of loading all had to be catered for ;
- The various points of entry of the cable into the ship introduced a plan range embracing an angle of 30°, i.e., 15° to either side of the axis of the gantry.

In view of the foregoing requirements and site conditions the structure was designed in three parts (as shown in Fig. 14), namely :

- A permanent bridge 346 feet long in four spans of from 70 feet to 100 feet, and 21 feet high above the nominal ground level, supported by two towers and three trestles ;
- Wire rope spans of 70 feet and 120 feet spanning from the tank house to the factory end tower ;
- Another wire rope span of about 100 feet from the wharf end tower to the ship.

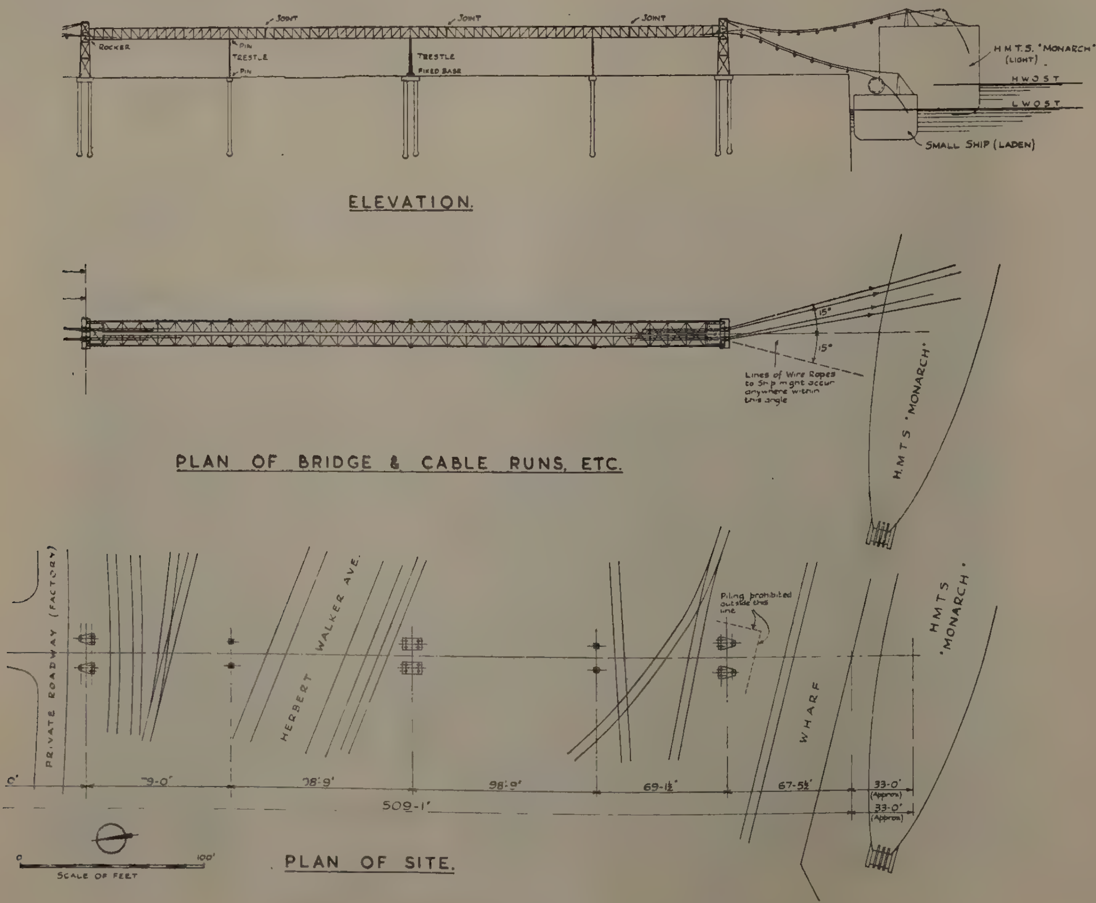


Fig. 14—General Layout of Installation

The two towers and three trestles are supported on piles about 40 feet long, surmounted by reinforced concrete pile caps.

The bridge is designed to span continuously over the five supports in the interests of economy, but as a precaution against any uneven settlement of the foundations (although this might be considered unlikely with piles) joints with slotted holes are introduced at three of the points of contraflexure.

The other four supports allow longitudinal movement of the bridge—the two trestles are pin-jointed at both top and bottom (although they are cross braced in the other direction to resist lateral forces), and rockers are used at each end for bearing on the towers.

In view of the longitudinal movement permitted, all longitudinal forces from wind and surge are resisted by the centre trestle, but each of the trestles and towers resist side wind.

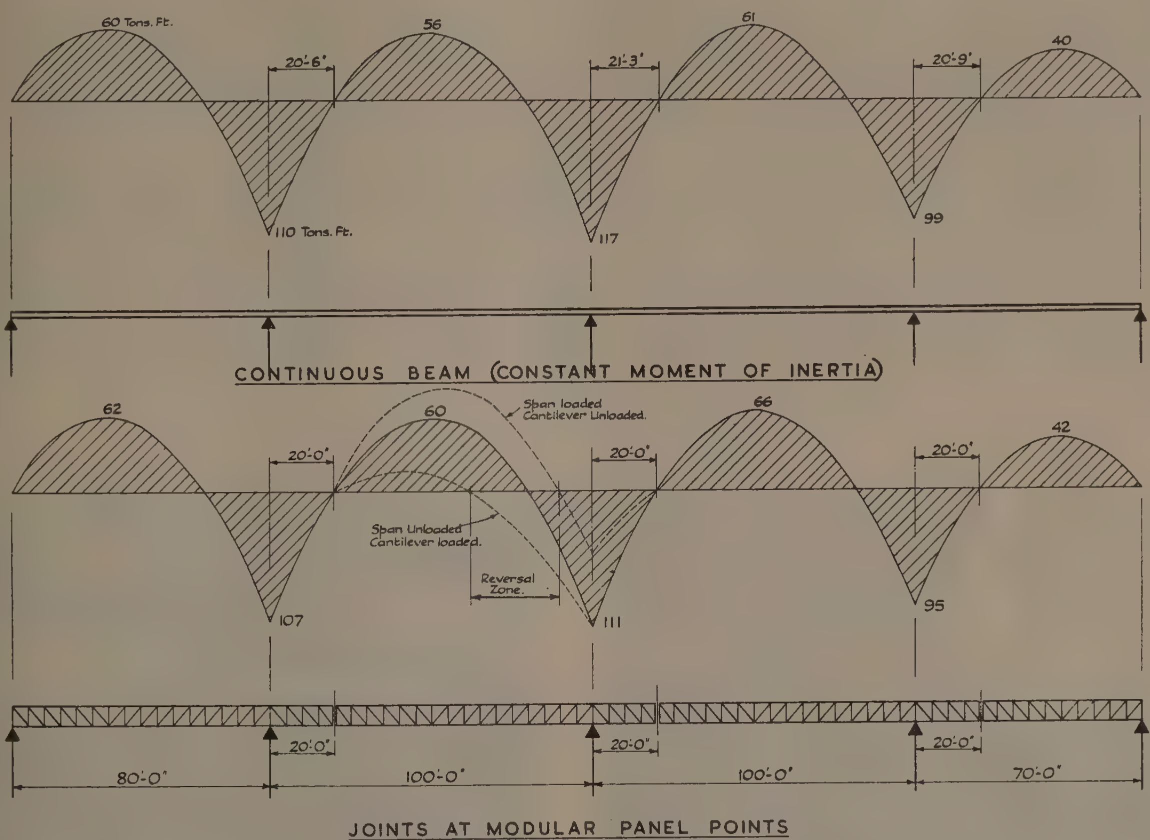


Fig. 15.—Continuous Beam Principle of Design

It was possible, within the limitations of the site, to proportion the spans to give very consistent bending moments, as can be seen in Fig. 15a, which is based on a continuous beam of constant moment of inertia. However, in the interests of repetition it was obviously necessary to standardise the spacings and panel points close to, but not quite coincident with, the true points of contraflexure. These positions of the joints slightly shortened the cantilevers (by amounts of 6 in., 1 ft. 3 in. and 9 in.) but Fig. 15b shows that this actually resulted in an even greater uniformity of bending moment. These diagrams are based upon uniformly distributed superimposed loading, but, of course, the occurrence of super load on one span while the next remains unloaded causes changes in the positions of contraflexure where the girders are not jointed and reversal of stress in some members. Fig. 15b illustrates one typical case of this.

The relatively short spans and the continuous principle of design enabled a shallow depth to be adopted for the main girders; therefore top wind bracing is omitted. The bottom only is braced, and the top booms of the main girders are stayed from the floor beams.

To cater for expansion and contraction the bridge is fixed only to the centre trestle, which is designed to cantilever about its base in each direction (Fig. 16).

The towers are each constructed of four R.S. Joist stanchions with bracing and torsion bracing, all similar in principle to those for Erith but, of course, very much lighter on account of the much reduced height and the much smaller spans and tensions of the wire ropes. The centre trestle comprises two stanchions and cross-bracing. The stanchions are of tapered joist section, fabricated from three flats welded together, with large gussetted bases the tops of which are sealed to prevent them from filling with water. Each of these two stanchions have four $2\frac{1}{2}$ in. diameter holding-down bolts to resist the considerable overturning moment from the longitudinal forces acting on the entire structure.

The main girders of the bridge are of light welded construction, factory assembled into panels 30 feet to 40 feet long. They are constructed of tee top and bottom booms, the angle internal members being welded directly to the tees with very small gusset plates forming an extension to the tee stalk, where required. The panels were site assembled with turned and fitted bolts.

The design proved exceedingly economical, the weight of steel in the 350 foot bridge, the two towers, and three trestles, being only 52 tons 11 cwt. This favourable weight is due partly to the continuous

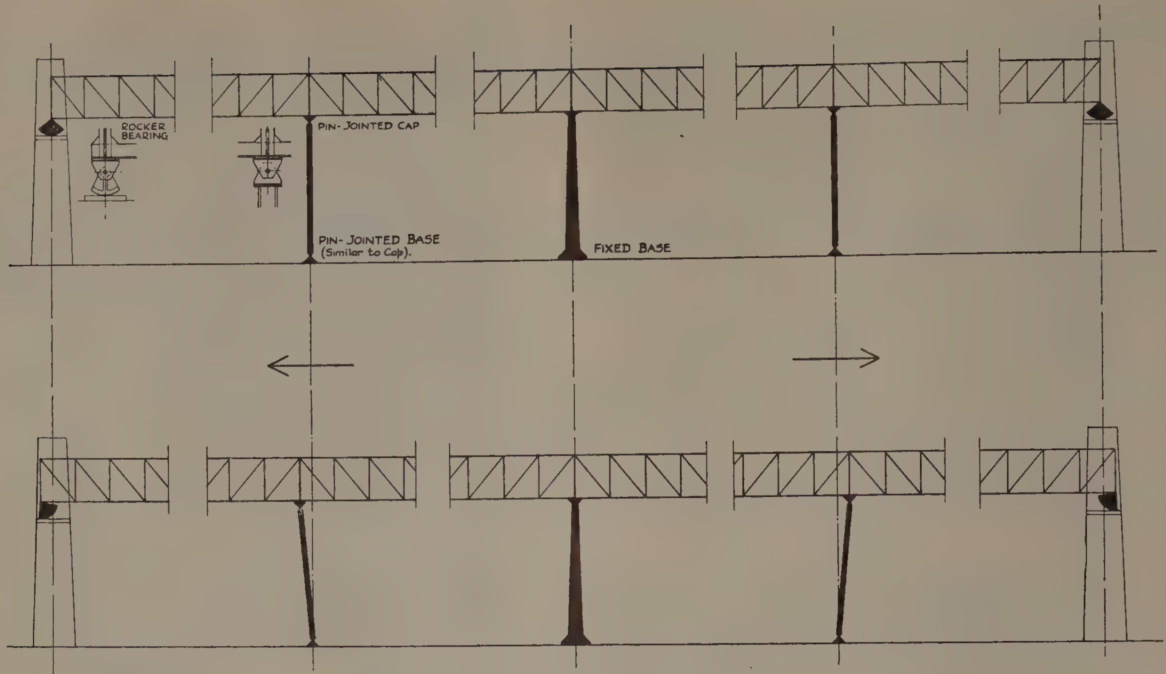


Fig. 16.—Provision for Expansion

(*By courtesy of British Transport Commission*)

Fig. 17.—First Section of Bridge Assembled on Ground and Awaiting Lifting

(*By courtesy of British Transport Commission*)

Fig. 18.—First Section of Bridge in Position

(*By courtesy of British Transport Commission*)

Fig. 19.—Completed Structure

principles, partly to the omission of top bracing, and partly to the welded construction of the main girder panels.

During erection advantage was taken of the railway tracks. The towers and trestles were erected normally, the pin-jointed trestles being stayed. After erection of the towers and trestles the first 100 feet of bridge, i.e., the first span and 20 feet of cantilever, was assembled on the ground (Fig. 17), complete with all bracings and framing, and lifted onto its supports bodily by means of a railway crane (Fig. 18). The next 100 feet of bridge was then handled in a similar manner, followed by another 100 feet and the final 50 foot section. Fig. 19 shows the bridge erection completed.

Foundation Problems

Every engineer is very conscious of the problems and difficulties that are liable to creep into foundation design, often very unexpectedly, especially in town sites.

Old drains and culverts, local soft spots from old filling, and projecting footings of adjoining property, commonly arise and create problems and, of course, the inevitable well must coincide with one of the main stanchions.

One project on which site conditions proved particularly unco-operative was close to a river which had been enclosed between thick masonry walls to form a culvert, the roof over which forms a main road through a town. At first sight the problem did not appear very unusual, the structure comprising a normal steel frame and reinforced concrete piled foundations. Bored piles were selected in order to minimise vibration that might have proved detrimental to the adjoining properties. It had been carefully ascertained from the local authority that the river wall was well clear of the site, hence no undue trouble was anticipated, (Fig. 20a).

another masonry wall returned from the river wall for several feet against the adjoining property (Fig. 20b). These circumstances necessitated a complete reconsideration of the structural design, but the scope of redesigning was severely handicapped by a number of practical considerations, viz.,

Firstly, any major change in plan or layout or construction of the building was out of the question because the majority of the piling was completed ;

Secondly, the structural steelwork was in hand, although fortunately not far advanced, and changes affecting material reservations would have proved a severe embarrassment to delivery ;

Thirdly, the river wall could not be cut into or disturbed, especially as girders occurred at close centres to carry the road, the bearing of the girders extending across most of the wall thickness and, more particularly, it was felt to be most inadvisable to interfere with the wall returning along the side of the site in view of the doubtful state of the adjoining property and its foundations, if any.

Finally, as might be expected, the client did not want the job to take any longer or cost any more.

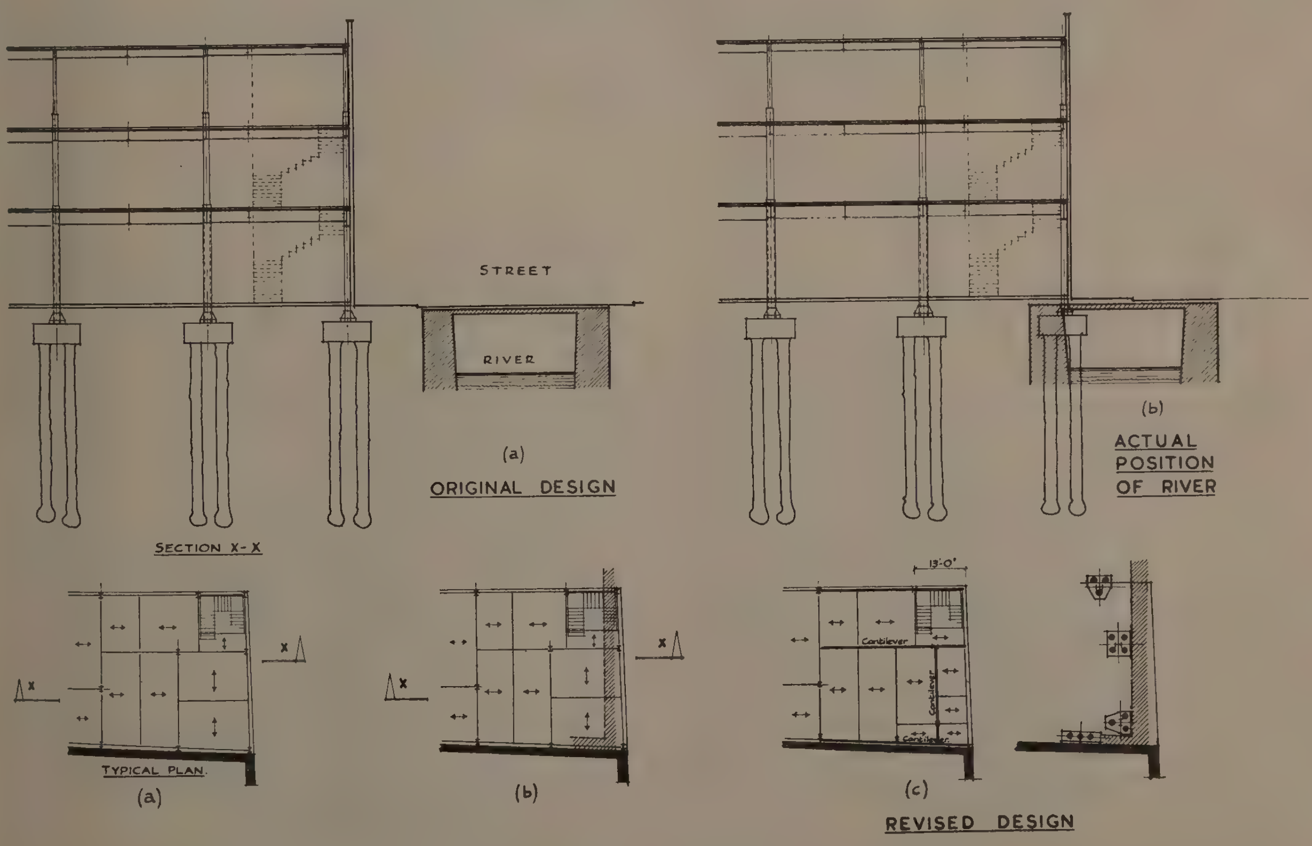


Fig. 20.—Original Structural Layout, Actual Position of River, and Revised Structural Layout

Piling operations were commenced at the rear of the site but unfortunately, after most of the piles had been completed, removal of old filling in preparation for the piles at the front revealed that the river wall had apparently "moved overnight" and encroached some 6 feet inside the building line, i.e., 9 feet inside the intended extent of the pile caps. Furthermore,

It was essential that the piles be brought inside both the river wall and the side wall, but the cantilevering of foundations was impossible on account of the extreme limitation in depth available (the beams carrying the road being only a few inches below the pavement level), hence the only possible course was to bring the stanchions into the building, over the pile

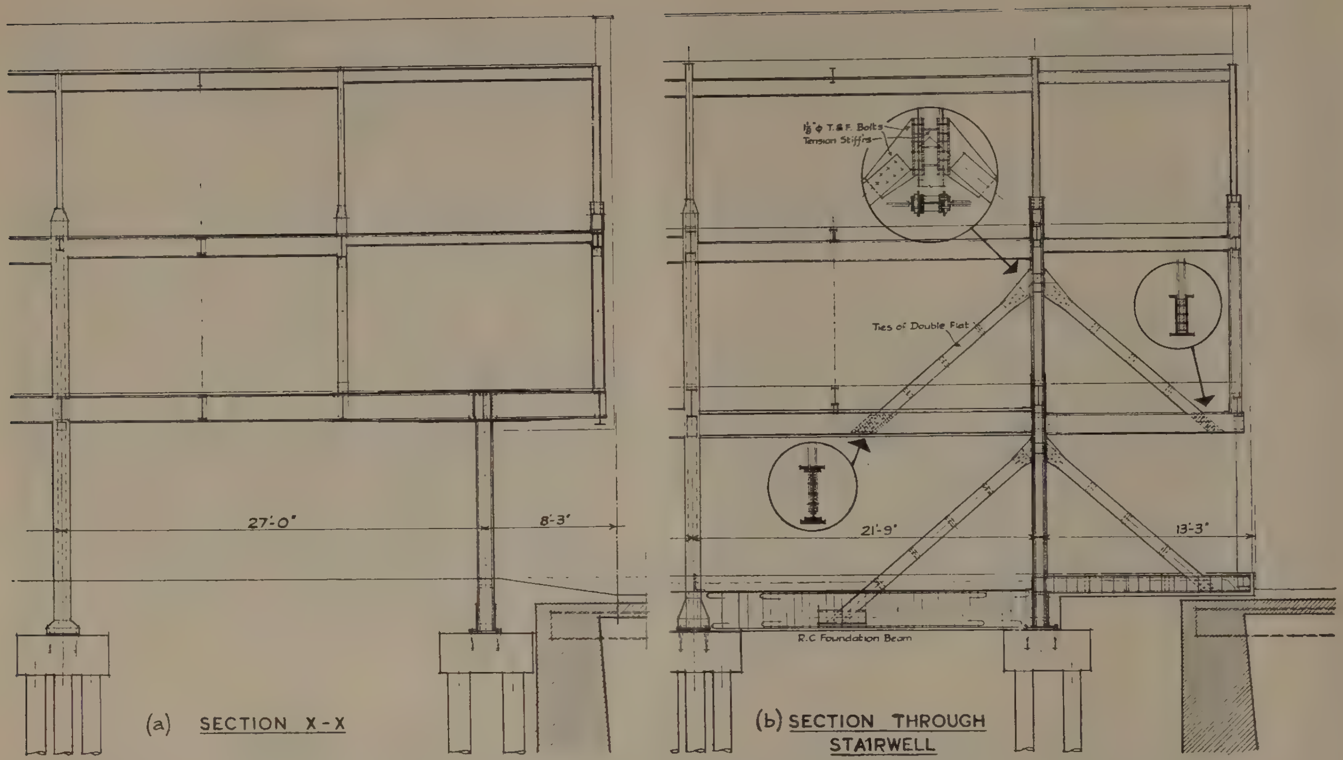


Fig. 21.—Sections Through Cantilevers

groups, and cantilever the first floor steelwork to the front and side (Fig. 20c). The projection of the front cantilevers is about 8 feet (Fig. 21a).

The next difficulty arose from the position of the main staircase at the front corner. Placing the corner piles and stanchion at the closest possible position to the river wall was impracticable because the stair well inhibited the use of ties at either the first or second floor levels, and such ties were considered quite indispensable, especially with a discontinuous stanchion. This stanchion therefore had to be taken back to the rear of the stairwell, and this resulted in cantilevering 13 feet. Even if a cantilever had been introduced at each floor level, the cantilever would have been rather heavy. In view of the unwieldiness of the 13 foot cantilever, also bearing in mind the limitations in materials available, it was decided to construct trussed cantilevers by inserting raking ties to the stanchions (Fig. 21b), the ties being of double flat section so that they could be embedded within the thickness of the cavity wall inner leaf. The same procedure was adopted for the wall foundation beams at ground level, the back tie then being anchored to the reinforced concrete beam.

All connections for the raking ties utilized turned and fitted bolts, $1\frac{1}{8}$ in. diameter. The original intention was for the bolts connecting the gussets to the stanchion to pass right through, but because of the practical difficulty of reaming the holes, separate turned bolts were used for each flange, and stiffener plates welded between the flanges to resist the tension across the stanchion.

Opening Roof

In introducing the last of the projects covered by this paper the author departs from the usual practice of describing jobs which have already been completed, and refers to one which has been deferred. He takes this course because he feels that the scheme is a particularly interesting one, and it concerns the erection of a large hall on an existing pier.

Fig. 22 illustrates the structure as existing. It comprises a reinforced concrete pier supported on piles, carrying a reinforced concrete cafe building at one end, another reinforced concrete building housing a stage and bandstand, etc. at the other end, an auditorium measuring approximately 130 feet long by 124 feet wide occurring between the two, and being surrounded by a promenade. The middle half of the auditorium area is open, and the remaining quarters of the area at each side are covered by canopies constructed of reinforced concrete cantilevers supporting almost flat timber and felted roofs. Each reinforced concrete cantilever occurs over a group of piles in the pier construction. The requirement was to cover the present open area with an opening roof so that, while the amenities and advantages of a permanent hall are available for bad weather and winter use, the open-air features of the present construction remain available for use during fine weather.

Obviously, the paramount consideration was that the construction be within the load-carrying capacity of the pier structure, any modifications to which would have been highly undesirable and very expensive.

The covering of the middle half only of the auditorium area, retaining the existing canopies, was not practicable, because the existing reinforced concrete cantilevers would not be strong enough to support at their tips the additional roof load that would be imposed upon them and, even if they were, the additional overturning moments at the bases of the cantilever columns would be so great that the piles would be

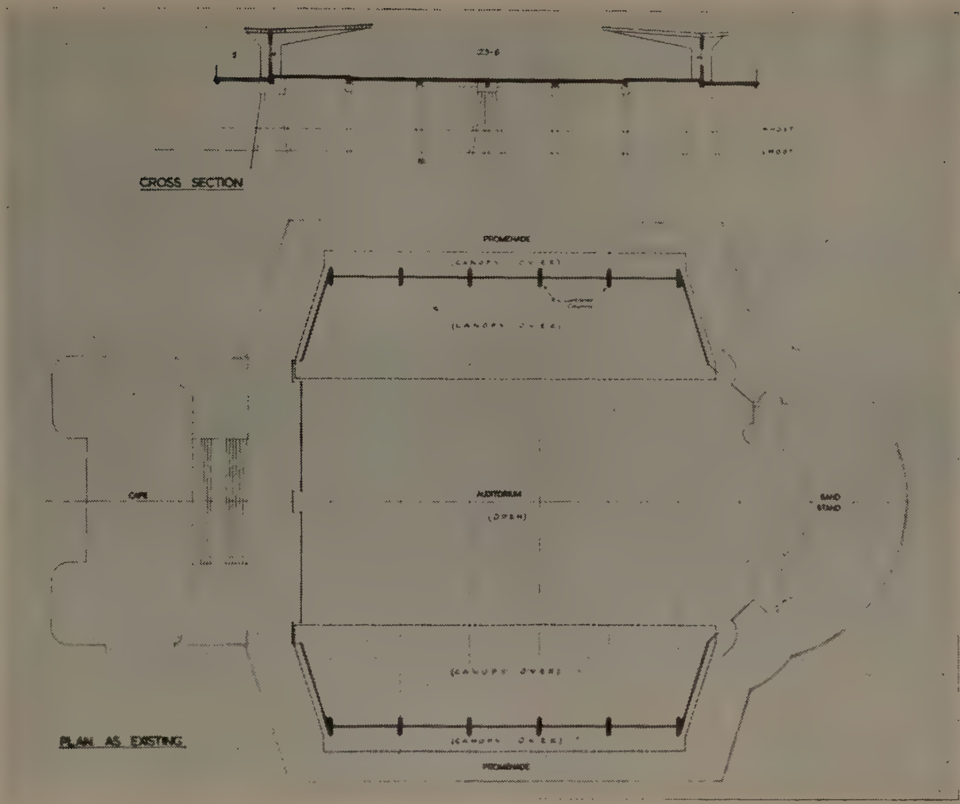


Fig. 22.—Plan and Cross Section of Pier Structure as Existing

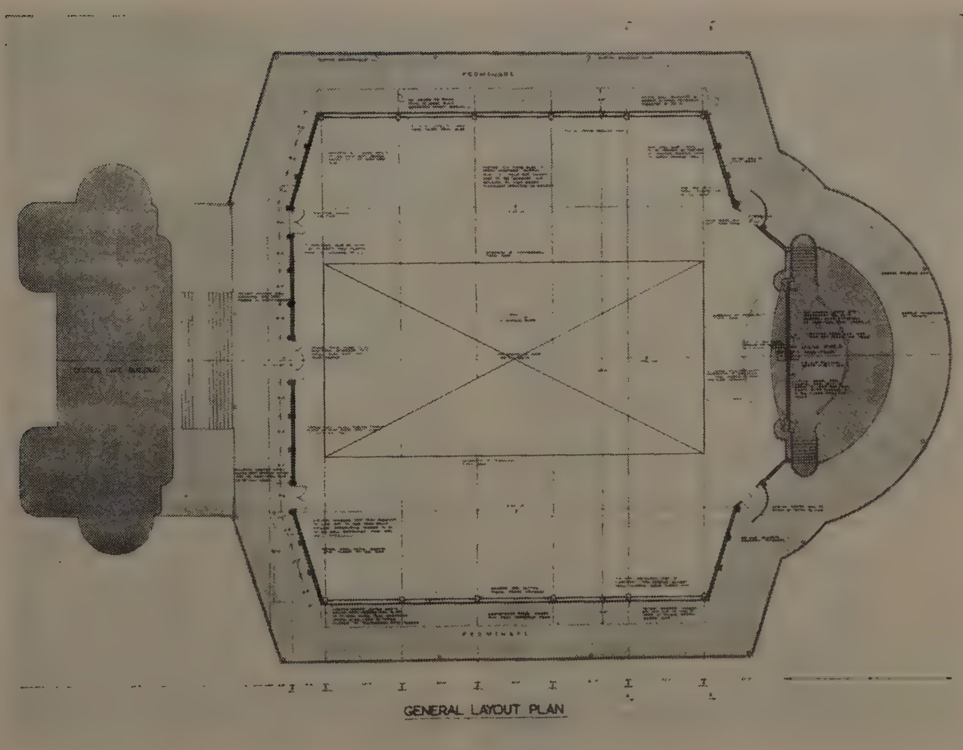


Fig. 23.—Layout Plan of Reconstruction

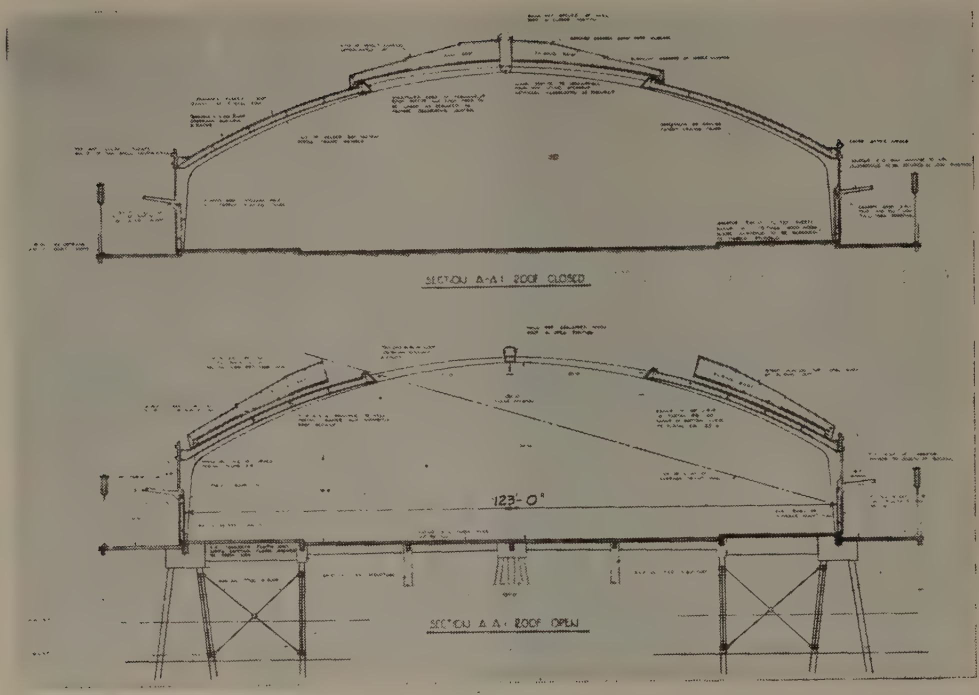


Fig. 24.—Cross Section of Reconstruction With Roof Closed (Top) and Open (Bottom)

overloaded. This could have been overcome by inserting twelve columns, one under the tip of each cantilever, but these columns would have caused obstruction of both view and movement in what would otherwise be a very fine clear floor area. Quite apart from these difficulties, the appearance of a roof incorporating large areas at such a low level was felt to be aesthetically bad.

The scheme therefore comprises the demolition of the existing canopies and their replacement by a new building of clear span and suitable height.

The new building is designed as a three-pinned portal frame structure of light construction, but with walls and roof of very high insulating properties.

The main area of the middle half of the roof comprises ten opening sections, designed to slide over the outer quarters of the roof, which are permanently covered (Figs. 23 and 24).

The rafters of the portal frames form a circular arc, firstly to give loftiness to the roof without unduly increasing the eaves height, and secondly to facilitate the sliding operation which demands either straight or uniformly curved rafters. Portal frames were selected to eliminate the considerable overturning moments that the present cantilever canopies transmit to the supporting piles, and a tie has been introduced below the floor surface between the two shoes of each portal frame to react the horizontal thrusts and avoid these being transmitted to the pier structure.

The aim was to avoid increasing the load transmitted to the pier by the new superstructure. A comparison of the weights of the existing and the proposed superstructures, omitting the weights of the piles, pile caps, beams and floors (which are common to both) shows that through the lightness of the proposed construction the weight transmitted to each pile group is actually less than under the existing construction, but when the additional effects of the maximum possible overturning moments of the canopies and wind are taken into account as well, the maximum load on any one pile is reduced to about a third of its present maximum, in spite of the increased area of roof directly supported by the particular groups of piles. This is due partly to the lighter form of construction and partly to the difference in superimposed loading stipulated for sloping and flat roofs in Codes of Practice.

On the other hand, the increased height of the building results in a greater surface area being exposed to wind forces, and cross braces are therefore introduced between the pier piles to form braced trestles in the four corners, the solidity of the reinforced concrete floor being adequate to transfer the wind forces to these four trestles.

The portal frames (Fig. 25) are of tapering box section, of two web plates and two flange plates welded together, with pins at the crown and attaching the portal frames to cast iron bases. The box section is introduced firstly against torsional tendencies from the sliding sections, and secondly to reduce lateral instability where ties are widely spaced to avoid obstructing the open areas.

Under the existing construction the occurrence of walls between the canopy columns ensures a degree of spreading of the superstructure load over the length of the pier on each side, and therefore of uniformity in loads on the pile groups. The lighter type of wall construction in the new building cannot be regarded as capable of spreading the load in the same way; hence a line of lattice girders is introduced on each side of the building between the portal frames to act as spreaders, and as a precaution against uneven settlement of the pier structure. Cross bracing on each side could well have been used for this purpose, but for the interference which would have caused to the side clerestorey windows.

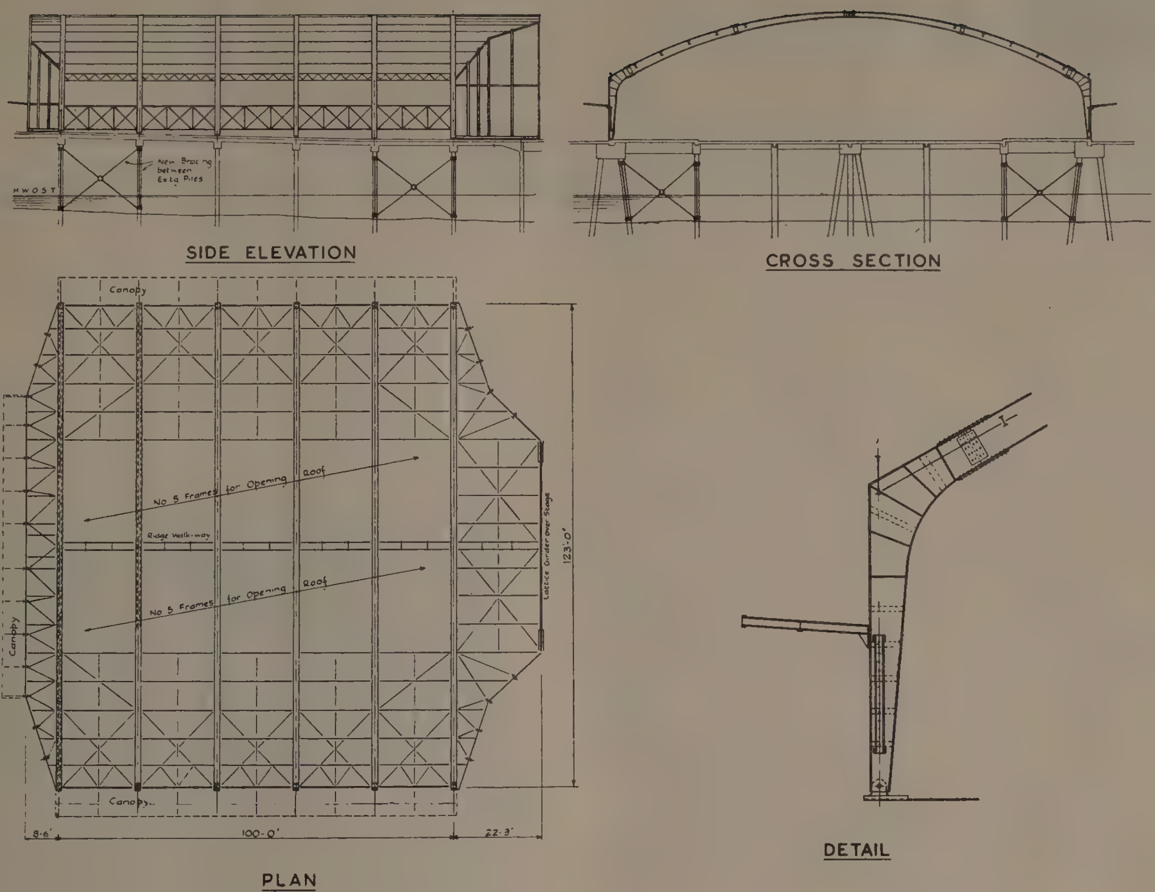


Fig. 25.—Plan and Sections of Steel Frame

The stanchions framing the end walls are also mounted on spreader beams for the same reason.

The walls are of cavity construction, and although of very light weight they have a thermal transmittance value of approximately 0.10 B.Th.U./hour/sq.ft./°F. difference in air temperatures, which is approximately three times more insulation than an 11 in. brick wall, for which the thermal transmittance value is 0.30.

The thermal transmittance value of the roof and ceiling combined is 0.37 B.Th.U./hour/sq.ft./°F., which compares very favourably with traditional constructions ; a tiled roof and plaster ceiling having a "U" value of about 0.90 and a solid concrete asphalted flat roof, plastered inside, having a "U" value of about 0.56.

The ten opening sections of the roof are each constructed of a steel frame with roof covering similar to the main roof, sides clad with boarding and the same kind of roofing felt, and a ceiling similar to that for the main roof. The roof of each section incorporates a large area of patent glazing to illuminate the building by daylight when the roof is closed, and at ceiling level under each area of roof glazing are two double-glazed laylights. Cold-rolled steel and light alloy were considered for the frames of the opening sections, in view of their lightness, but were not adopted because of the potentially corrosive action of the maritime atmosphere.

The dimensions of the roof openings and the sliding sections are proportioned so that the opening sections are completely hidden from view from inside the auditorium when the roof is fully open. To avoid the ridge presenting a clumsy appearance when the roof is open it is designed to collapse to a relatively shallow depth—the collapse of the ridge also eliminates the necessity for cladding its two sides.

The whole of the mechanism is controlled by two switch units ; one operates the ridge assembly and the other operates the ten opening sections. The two switch units are fully interlocked electrically to ensure correct operating procedure. The total running time required for either the complete opening or complete closing operation is about two minutes—just under one minute for the ridge and a little over a minute for the sliding sections, and the design incorporates three independent forms of brake or safety device.

Acknowledgments

In closing the author wishes to acknowledge the permission of Messrs. Submarine Cables Ltd. to describe and illustrate the Cable Loading Structure at Erith ; Messrs. Standard Telephones & Cables Ltd. for similar permission respecting the Loading Installation at Southampton ; Messrs. Sheppard & Sons Ltd., the steelwork contractors for those two projects ; the Editor of *The Times* for permission to reproduce three of the photographs of Erith, and The British Transport Commission in connection with the erection photographs of Southampton.

Some Notes on the Use of High Preload Bolts in the United Kingdom*

Discussion on the Paper by F. M. Easton, A.M.I.Struct.E., A.M.I.C.E., E. M. Lewis, A.C.G.I., A.M.I.W. and D. T. Wright, B.A.Sc., M.S., Ph.D.

THE CHAIRMAN introduced the authors, Mr. Easton and Mr. Lewis, who presented the paper.

Dr. Wright, who was in Canada, was unable to be present and had sent a tape recording of his introductory remarks. Discussing some of the more recent developments in the use of high strength bolts in North America, he suggested that the notion of a bolted friction joint had now been generally accepted by structural engineers everywhere. It had been repeatedly demonstrated that a bolted joint was at least as good as a riveted joint, provided that the bolts were properly tightened. The most serious problem with respect to the practical use of high strength bolts lay in securing required bolt tensions under field conditions. When the first specification of the Research Council on Riveted and Bolted Joints was written in 1951, it was suggested that 5 to 10 per cent of all bolts should be loosened off and retightened with a torque wrench to check tensions. This, of course, was based on the presumption that the bolts would be tightened by hand with ordinary wrenches. With larger diameter bolts, and particularly when working in tight quarters, it was found almost impossible to tighten bolts adequately with simple hand wrenches. Further, with bolt tightness dependent on a workman's " sense of feel ", even 10 per cent inspection was not always adequate.

Almost as soon as the bolts were used practically, pneumatic wrenches were used for tightening, and of course proved to be much faster and more convenient than the hand wrench. It was at first hoped that some simple general relation could be found between air pressure and bolt tightness. Results of tests were too erratic, however, and torque wrenches had to be used for inspection as with simple hand wrenches. A second approach had proved successful, however. A typical relationship between bolt tension and time for various air pressures was then illustrated. It had been found that by controlling air pressures close to the wrench—within about 10 feet—and by measuring the time of operation, quite uniform torquing efforts could be obtained from a given wrench. The next illustration showed a device for calibrating wrenches by measuring tensions directly at the site. Very good consistent results were obtained by putting a bolt (of the same size, and from the same lot as those to be tightened) in the device and adjusting an air pressure regulator until, say, 10 seconds effort gave the desired bolt tension. The device in detail was next illustrated. There was still some scatter—perhaps 5 or 10 per cent—but this was compensated for in practice by tightening the bolts to an average tension somewhat greater than that required. The method had the obvious advantage of eliminating torque wrenches—which were by no means free of scatter themselves.

A direct method of obtaining desired torques was available with pneumatic wrenches sold by one company, and which had torque bar controls which cut off the air supply when the desired torque was reached. Tests had shown these wrenches to be very satisfactory.

An alternative development appeared to offer great advantage in simplifying the field bolt-tightening operation, and eliminating all measuring gauges and such from the site. It had been observed that to give a nut a half-turn produced a tension almost up to the minimum specified value. After one full turn, there was little increase in bolt tension, but to break the bolt or strip the threads required 2½ to 3 full turns. Tests had shown that joints in which the bolts had been tightened almost to breaking would function satisfactorily, and that if the bolts did not fail while being tightened they would not fail at all. It followed then that to give a nut a full turn from a finger tight position would produce a sufficient yet safe bolt tension. That held true for all bolt diameters and almost all grip lengths. In use this was very simple since the operator need only assure himself that the material to be fastened was in good solid bearing then run the nut up with his finger, and use the air wrench until the chuck made a complete revolution. If the parts to be joined were not well fitted up, a few bolts could be temporarily tightened then later loosened and retightened properly.

In Canada, the turn-of-the-nut method was now being used very commonly. In the United States, the American Bridge Company still used calibrated impact wrenches, while Bethlehem Steel Company, the other major fabricator, used the turn-of-the-nut method.

Although the superior strength of the high strength bolt, particularly in fatigue, led to its use first in highly stressed members in railroad bridges, the remarkably extensive use of high strength bolts in North America had been due most of all to their economy as a field connector. Rivets were still most economical for shop fabrication, but the ease with which bolts could be installed made them ideal for field work.

Dr. Wright concluded by remarking that the original American notion of the substitution of a rivet by a bolt of the same diameter was being modified. For steady loading, as in buildings, higher loads were now permitted on bolts, and it seemed that with the advent of super-strength bolts, capable of carrying higher tensions, even greater efficiencies would be possible.

MR. EASTON, introducing the paper, referred to more recent experience on the Western Region in applying the code of practice summarised on page 170, but with

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1., *on the 24th October,* 1957. *Professor Sir Alfred Pugsley, O.B.E., D.Sc., F.R.S., M.I.Struct.E. (President) in the Chair. Published in " The Structural Engineer," Vol. XXXV, No.* 5, *pp.* 167-175. (*May* 1957).

the reduced torque values mentioned in the fourth paragraph of page 171. When screwing up the bolts with a torque controlled impact wrench, the mild steel nuts continued to fail occasionally, more particularly when tightening was effected in from three to five seconds per nut. This speed was, admittedly, unnecessarily high, but the failures, although few in number, did raise doubts whether the nuts could be regarded as generally satisfactory.

In the first place, failure had resulted from the nut thread stripping ; a sheared-off spiral of thread (illustrated in Fig. 13) would be formed by progressive failure from the nut face inwards as the tightening proceeded, the spiral remaining embedded in the bolt thread, which was undamaged. Secondly, there was a tendency for the nut to "bell out" under the load, that is, to expand radially at the bearing face.

Fig. 13.

In consequence of these doubts, it was decided to use nuts of 'P' quality steel to B.S.1083 (35/48 ton/sq. inch) in place of the mild steel nuts formerly specified, and to revert to British Standard Whitworth threads. The prescribed tensile loading on the bolts would be maintained, since the bolt steel could be relied upon to carry the increased stress resulting from the reduced "stress area" of the thread.

An examination of Fig. 14 would show the reasons for this decision. In the lower part of the figure, $\frac{7}{8}$ in. diameter bolt and nut threads, both B.S.W. and B.S.F., had been drawn to scale with the maximum tolerances prescribed under B.S.1083, the threads being shown in the relative positions they would occupy when screwed up under load. Obviously, with the B.S.W. thread, there was considerably more contact between bolt and nut than there was with B.S.F. thread ; and the B.S.W. would be stronger, even allowing for the increased pitch. In addition, the thread loading was not uniformly distributed throughout the depth of a nut ; owing to the elasticity of the material, a large proportion of the load came upon the first few threads. The approximate distribution was indicated in the right-hand part of Fig. 14, from which it would be seen that the concentration of load would be appreciably less on the coarse thread than on the fine.

Mr. Easton ended with an expression of apology with regard to the photograph of the prototype torque multiplying spanner reproduced in Fig. 8. That photograph had been put into the paper inadvertently without either the permission of or acknowledgment to the firm responsible for developing the tool which, in its final form, was a remarkably light, compact and efficient instrument for tightening spline extension bolts.

Mr. E. M. Lewis said that since the paper was written he had been concerned with experiments to further the application of the half torque half turn technique or, as the authors would now prefer to call it, the part torque part turn technique. Fig. 15 was typical of the results obtained with bolts of normal grip length and also for holding down studs. The dotted lines indicated the limiting behaviour when the part torque was one quarter and three quarters of the nominal torque required to tighten the bolts using the torque coefficient technique. It would be seen that the "half torque" normally specified need not therefore be applied with any great degree of accuracy as the variation in the bolt load after approximately three quarters turn of the nut was only about 7 per cent. The variation before applying the three quarter turn of the nut was about 200 per cent. The curves also demonstrated that thread lubrication which was so critical when the torque coefficient method was used, no longer caused significant variation.

From the results which were at present available the tightening rule which had been evolved was as follows : after the nominal half torque had been applied an additional half-turn should be given for each nut plus a further quarter of a turn for each 6 in. of bolt length. Thus for bolts of 6 in. grip length, half torque and three quarters turn should be applied. For studs of similar grip length an additional half turn was allowed for the second nut making one and a quarter turns in all.

Mr. Lewis referred to two other current series of tests. Since joints subject to tensile load induced small variations in bolt tension, it was desirable to understand the fatigue behaviour of the bolts under high preloads. Tests were in hand on bolts in B.7a (B.S.1506), En.110U and En.16R (B.S.970) grade steels to examine the fatigue behaviour at prestresses in the range .6 to 1.2 times the yield stress.

The other investigation concerned the use of certain types of plastics to increase the efficiency of the joint. It was hoped to develop a coating which would protect the steel from corrosion prior to erection and which would, after erection, greatly improve the slip coefficient of the joint thus enabling the number of bolts and the total preload to be considerably reduced. This "semi-glued" joint might not be as revolutionary as it sounded when it was realised that Dr. Dornen in Germany had already built, with the exception of service bolts, a completely glued all-steel bridge.

Discussion

The President said it had been suggested that this country had been behind others in regard to the matter under discussion. But if we looked back over the history of the problem in this country we found that once or twice the solution was very nearly reached ; we had approached the pre-tensioned bolt, as it were, and had then sheered off ;

The first time he could remember that happening was when Professor Unwin had shown an interest in the stresses occurring in bolted and riveted joints. In

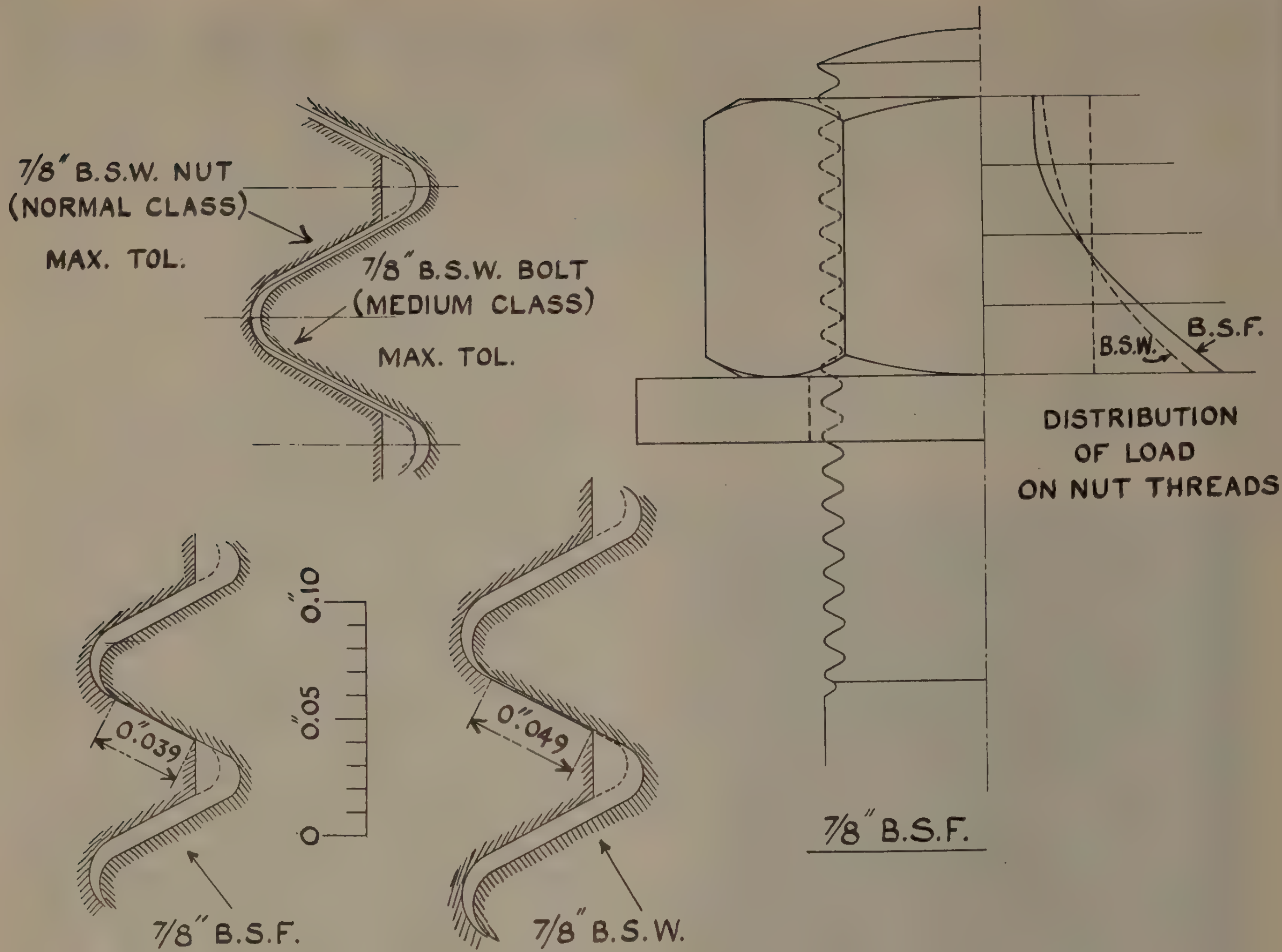

7/8" B.S.W. NUT
(NORMAL CLASS)
MAX. TOL.
7/8" B.S.W. BOLT
(MEDIUM CLASS)
MAX. TOL.
B.S.F.
B.S.W.
DISTRIBUTION
OF LOAD
ON NUT THREADS
7/8" B.S.F.
0.10
0.05
0
0".039
0".049
7/8" B.S.F.
7/8" B.S.W.

Fig. 14.

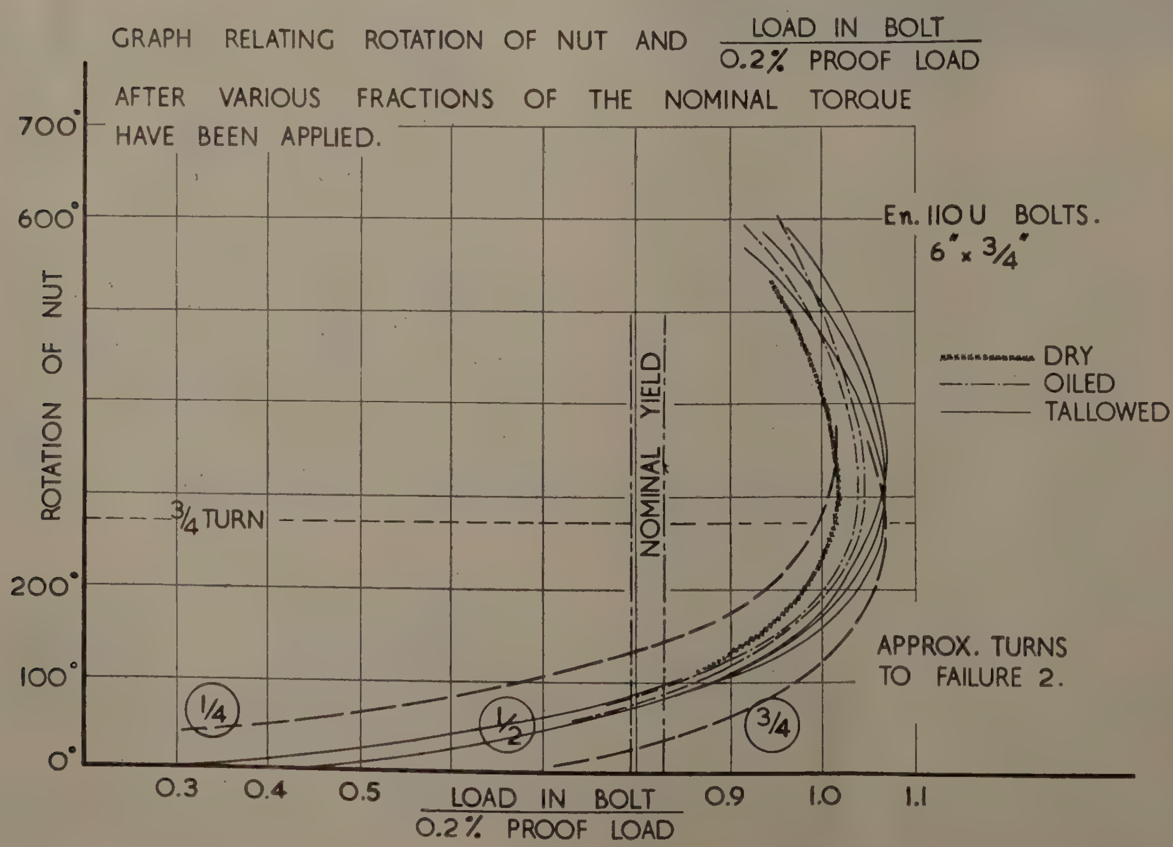

GRAPH RELATING ROTATION OF NUT AND LOAD IN BOLT / 0.2% PROOF LOAD
AFTER VARIOUS FRACTIONS OF THE NOMINAL TORQUE
HAVE BEEN APPLIED.
700°
600°
200°
100°
0°
ROTATION OF NUT
3/4 TURN
NOMINAL YIELD
En. 110 U BOLTS.
6" x 3/4"
DRY
OILED
TALLOWED
APPROX. TURNS
TO FAILURE 2.
1/4
1/2
3/4
0.3
0.4
0.5
0.9
1.0
1.1
LOAD IN BOLT / 0.2% PROOF LOAD

Fig. 15.

his early book on "Elements of Machine Design" he had given an account of some continental work on such joints, which assumed that their behaviour, even without careful control of the tension, was better discussed in terms of friction than in terms of shear. But nobody had taken much notice of it.

The second occasion when we had come near to a solution was about 1930, when there was a Steel Structures Research Committee, of which Professor Batho was a member. He did many tests on the behaviour of bolted steel structures and made an effort to introduce into their analysis the effect of friction at the joints. Again, for reasons which many of us knew, we had left that alone.

The President then proposed a hearty vote of thanks to the authors for their paper.

(The vote of thanks was warmly accorded).

Mr. P. S. A. Berridge (British Railways) said he could claim to have been behind the scenes with the authors during the writing of the paper and he was indeed grateful to the President for allowing him to join in the discussion.

He made no apology for using the term "High Strength Bolt." It was the name by which those bolts had been known in the United States and it was the name by which they were known on British Railways and by many bolt makers, contractors and others in

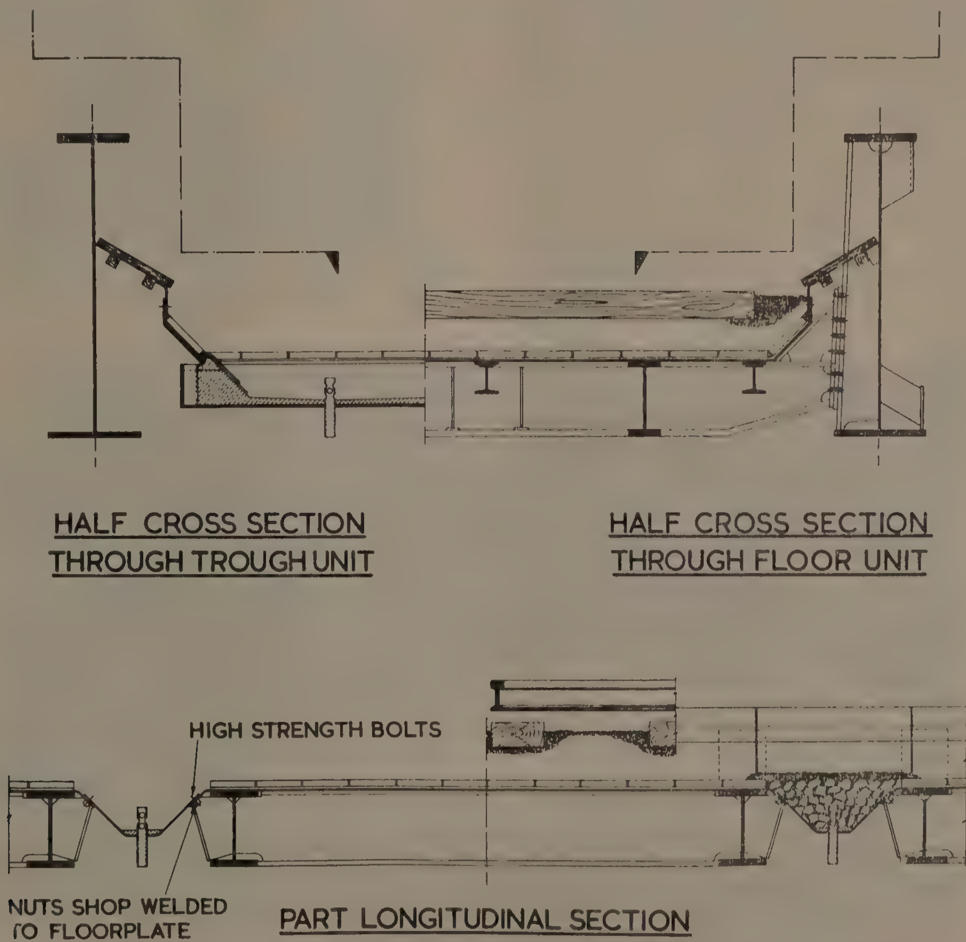


Fig. 16.—The design of floor for half through type plate girder spans standardised on the Western Region of British Railways.

Now, following the work of the authors of the paper, we had another chance of getting ahead.

The question of "glues" had been mentioned by Mr. Lewis. At the moment "glues" intended for sticking metals were being studied near Cambridge, and at Cambridge itself there was also another development on which perhaps the authors would comment. When we did experiments on frictions at school we found it difficult to believe the so-called laws of friction, but now this whole matter had been placed on a scientific foundation by the work of Bowden at Cambridge. As a result, we were now ahead of the rest of the world in this field. Could the authors make some remarks on possible applications and developments from this?

this country; and frankly he was sorry to see the authors trying at this late hour to change a name which had already become a household word in the structural engineering industry.

High strength bolts were first used on Western Region railway bridges some six years ago; that was the first time they had been used on a railway bridge outside the United States of America. The high strength bolt was adopted as a connector which, unlike the rivet, could be *properly* used in *direct* tension.

The modern trend in bridge design was to make the best possible use of prefabrication in the shops. In the half through type welded plate girder bridge the floor almost always had to be connected to the main girders

on site. (An exception, of course, was the Twickenham flyover bridge described about a year ago by Mr. Palmer).*

The design developed at Paddington consisted of deck units of either precast prestressed concrete or prefabricated steel. In the latter, each unit consisted of a pair of cross girders with stringers and floor plating all shop-welded and waterproofed before delivery on site. Shear plates on the ends of the cross girders were landed on similar shear plates welded on the sloping flanges of tee-shaped stiffeners, while the bending moment stresses at those joints were carried entirely by high strength bolts in direct tension. Fig. 7 in the paper showed a method of tightening those particular high strength bolts with a torque-limiting wrench used in conjunction with a torque-multiplying wrench, the latter giving a mechanical advantage of about 7 to 1. That form of construction permitted the decks under each track in a multi-track span with intermediate main girders to be quite independent of one another—a great advantage when erecting the first half of a double-track bridge while half the old structure was being retained for traffic. The rate of erection of the deck units was not dependent on the tightening of the high strength bolts; that could follow later on, so long as sufficient bolts were inserted in the holes to keep the shear plates in contact while traffic was allowed across at slow speed.

had followed and those bolts were now widely used for all repairs to dynamically loaded girder structures on the Western Region. Faying surfaces were not painted, and with the high strength bolt developing twice the clamping effect on a good hot-driven rivet, ample friction was provided to carry the shear load across the connection.

The web and flange joints in the 170 ft. long main girders continuous over three spans in a railway bridge near Llanymynech had been made entirely with high strength bolts in clearance holes, correct alignment being made with parallel shank drifts in the first instance.

Pneumatic Impactools set to cut out at the predetermined torque were used to tighten those bolts. When the air pressure was constant and the air supply adequate, those tools worked quite well; but they were heavy and cumbersome and he was inclined to favour partial hand-tightening (say) to 200 lb.ft. torque and then giving each nut the requisite amount of turn to produce the required tension in the bolts.

That led Mr. Berridge to the Torshear bolt, shown in Fig. 20.

That really was, he said, a clever thing, and he understood it was being put into production by several bolt makers quite shortly. It was the high strength bolt

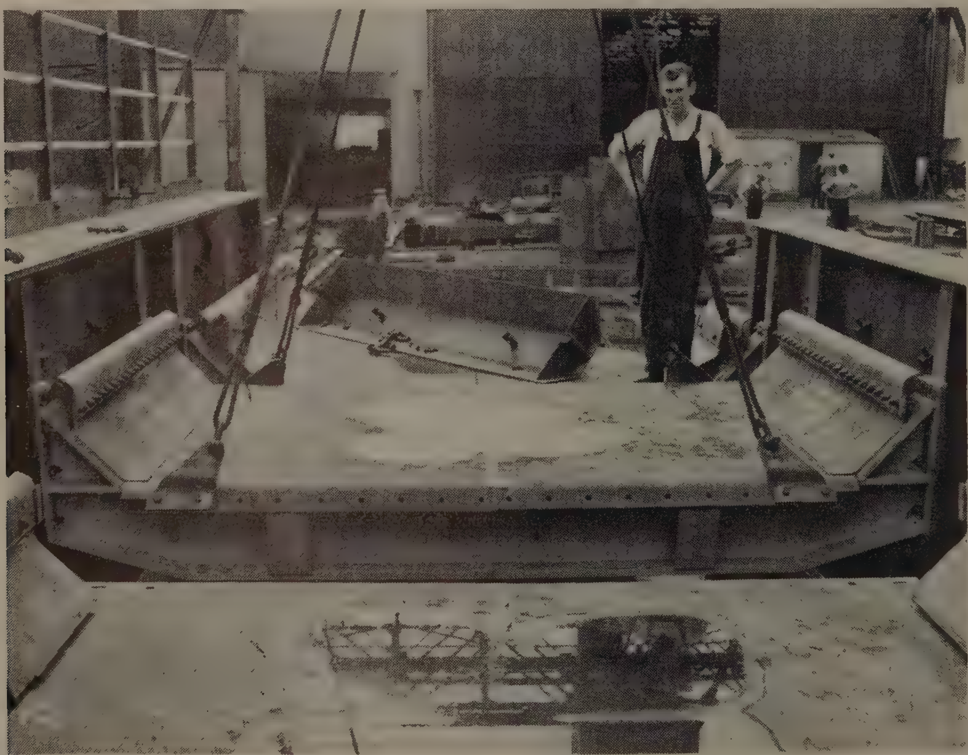


Fig. 17.—Slinging a prefabricated steel deck unit between the main girders, showing also a trough destined to span the gap between the units.

Single pressed steel trough units between the prefabricated steel deck units served two functions. They facilitated drainage of the ballast and through their flexibility they prevented the building up of high stresses in the deck units due to floor interaction.

So much, then, for the initial use of high strength bolts in direct tension. Their adoption to carry shear

* M. F. Palmer, "Fabrication and Erection of Steel Plate Girder Railway Bridges," "*Struct. Engineer.*" *Vol. XXXII, No. 12, p.* 322. (1954).

with an extension of the threaded portion beyond a groove. The Torshear bolt was tightened with a special pneumatic wrench fitting over the nut and gripping and turning against the extension of the threaded portion beyond the groove. The nut was turned until the extension broke off, by which time the bolt would have been tensioned to the required amount. Naturally, the groove must be made accurately; but that, he was told, presented no special problem. An inaccuracy of 0.002 in. in the diameter of the groove

Fig 18.—High strength bolted joint in main girders of the bridge near Llanymynech.

Fig. 19.—Tightening High Strength Bolts with a Pneumatic Impactool.

in a $\frac{7}{8}$ in. diameter bolt resulted in a variation of 10 lb.ft. in the torque. The great advantages of the Torshear bolt were that the wrench gripped the bolt : one could see at a glance whether or not a bolt had been properly tightened ; and it could not be over-tightened.

Mr. L. R. Creasy (Superintending Structural Engineer, Ministry of Works) spoke of the Ministry's experience of the application of high strength bolts to normal frame construction. They had been used on a few contracts of quite moderate tonnage, as shear connectors in lieu of site riveting and had been substituted on a one-for-one basis. Their cost was not more than that of site riveting and they had been

Fig. 20.—Torshear Bolts before and after tightening.

found to be infinitely more convenient and speedy to use.

They had used both manual tightening of the bolts which was somewhat slow and pneumatic wrenches in which the air was adjusted to a controlled pressure to provide a specified torque. This latter method was quite accurate and both speedy and convenient. In each contract there had been a small percentage of breakage of the bolts, usually under the nuts.

In order to arrive at a suitable technique for using these bolts they had conducted a few laboratory tests, and Mr. Creasy illustrated the results. The tests showed the relation between the applied torque in the bolt, the ultimate shear between the plates and the bolt tension for a range of values of applied torque. The test jig, illustrated in Fig. 21, consisted of two

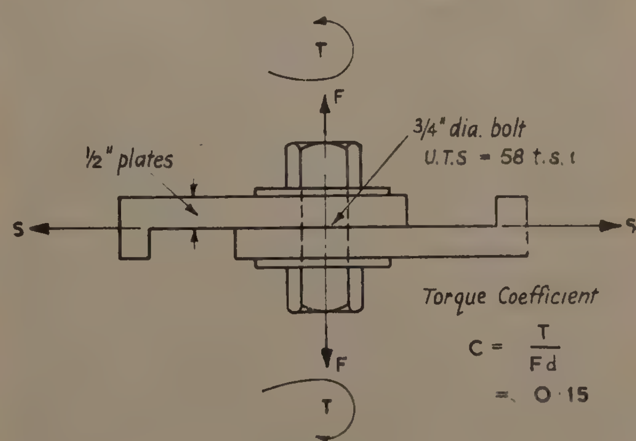


Fig. 21.

$\frac{1}{2}$ in. mild steel plates, commercially clean and connected by a single $\frac{3}{4}$ in. diameter high strength bolt with adequate clearance in the shank. In these tests the torque coefficient approximated to 0.15 which was less than that used by the Authors and was an indication of the variation to be found in practice in this important factor.

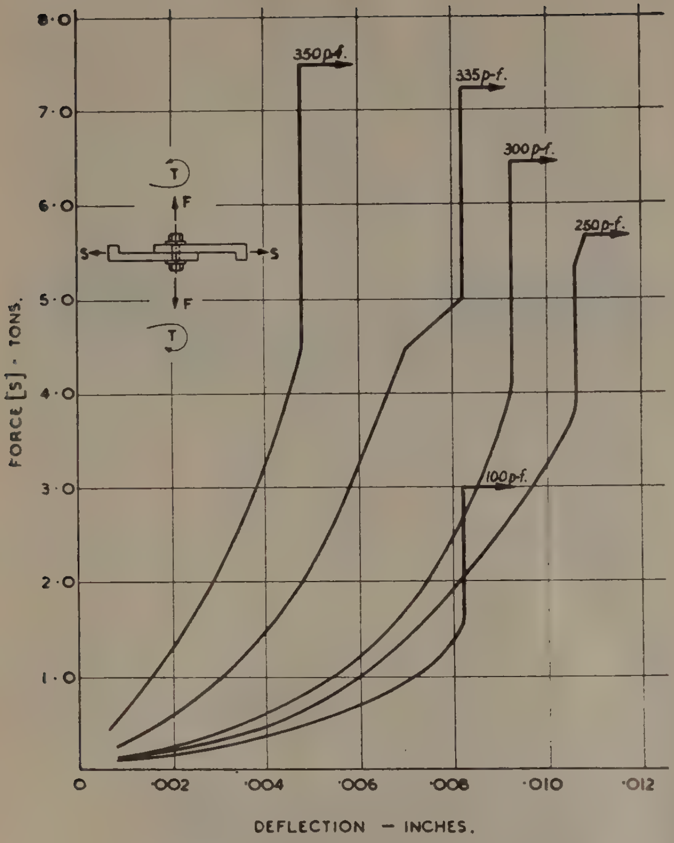


Fig. 22

Fig. 22 showed the relationship between an applied shear force to the connection and the relative lateral deflection of the plates for different values of applied torque. The curves were of similar type and indicated an initial period of deflection or slip between the plates, after which no additional movement was observed between the plates with increasing values of shear until final slip and ultimate failure.

The relation between the applied torque, the induced bolt tension and the ultimate shear in the connection was shown in Fig. 23. The lower horizontal line indicated the working load of 3.2 tons in shear for a shop rivet and high strength bolt of $\frac{3}{4}$ in. diameter. The dotted line indicated the zone during which secondary slip or deflection occurs in the connection. The straight line effect appeared to be maintained throughout the full range of bolt tension as far as failure although in fact a very small permanent set was recorded at approximately 85 per cent of the ultimate.

An applied torque of 300 lb.ft. had been selected for site working which was also the one recommended in the paper. It would be seen from the graph that at this torque the connection was within the region of initial deformation throughout the range of the working load. At this torque the ultimate shear in the connection showed a load factor of 2 over the design working load and the induced tension in the bolt represented a load factor of 1.1. on yield and 1.3 on the ultimate strength of the bolt.

The load factor of 2 in the working shear load of the connection seemed rather greater than was necessary and appeared to support the authors' recommendation that the nominal one-for-one basis of substitution of high strength bolts for rivets might be improved.

Concerning the relatively small load factor against yield and ultimate tension in the bolt the question arose as to whether a smaller torque should be used with a lower induced bolt tension. Of not lesser importance however than the breaking of the bolt was the shear resistance in the connection which was the fundamental design factor and must be maintained. In this respect it was extremely difficult on site to guarantee both the applied torque and the frictional resistance and variations in these might reduce the shear between the plates to possibly dangerous limits. Once the bolt was tensioned, however, no variation in the applied load on the connection could increase the bolt tension and a smaller margin of safety would seem justified in the bolt than in the induced shear in the connection. We were faced with a type of connection where, provided the bolt did not fail, this must be presumed as a positive load factor, however small, and as such must be accepted as satisfactory.

Mr. H. C. Husband (Hon. Curator) said he was very glad to have the opportunity to join in the discussion of such an important paper, which had been so fairly presented by the authors.

Commenting that he hoped his remarks would not appear to be retrograde or hopelessly reactionary, he went on to say that he had used high strength bolts, in particular for structures subjected to heavy vibration. He was not at all prejudiced.

The authors had said that we should not look upon high strength bolting just as an alternative to site riveting. But in respect of large jobs he thought that was how most of us were bound to look at the situation. Site riveting would become more and more difficult in the future as we had fewer and fewer riveters, and the position with regard to site welding was just as serious ; hence the use of the high tensile bolts would be extremely important in the future.

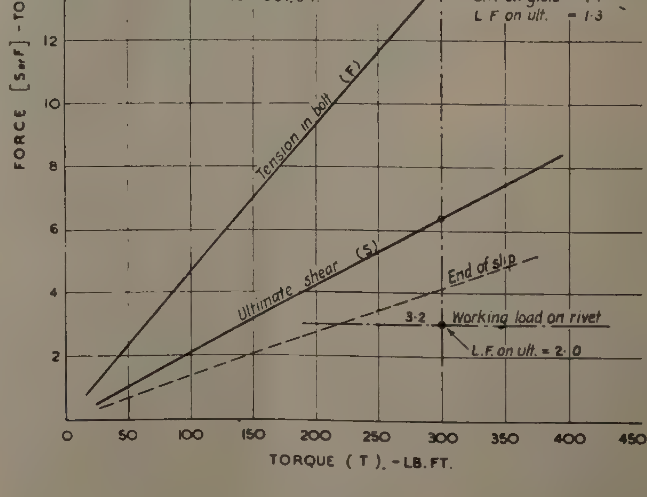


Fig. 23

In connection with one or two large jobs during the last two or three years his firm had been asked to consider the use of these bolts. On one job in particular a highly reputable firm of constructional steelwork contractors had asked them not to use the bolts because of the possible lack of reliability and lack of uniformity in them.

It was stated by the authors that on a certain occasion four out of 40 bolts had failed. One immediately wondered how near to failure the other bolts might be. Taking the ordinary job, the amount of careful control on the site was not all that it might be.

We all knew how difficult it was to make sure that tapered washers were fitted properly; that was another extremely important matter, and it was mentioned in the paper. It was important in the case of ordinary black bolts, but he was sure it was much more important when using the high tensile bolts.

As a comparison, where high tensile reinforcing bars were used, in pre-stressed concrete contruction, of considerable diameter, anchored by nuts, the greatest care was necessary to ensure that the bearing plates behind the nuts were precisely at right angles to the axis of the bolts. Most of us knew from experience that if the bearing plates were not at right angles to the axis of the bolt there was trouble, and the high tensile bar, taking up the load on the nut, cracked.

There was another analogy. From time to time, when there had been failures in cases of threaded high tensile bars, they had been attributed to imperfect thread cutting, a worn die chaser had possibly been used in finishing the threads.

Where high tensile bolts were used on a large scale on jobs where very close supervision might not be possible throughout the whole of the erection period we must have very reliable bolts. He was speaking as a Sheffielder, and in Sheffield they were interested in the production of uniformly high quality steel. On the other hand, we all knew how difficult it was to achieve this ideal.

The American specification quoted by the authors (in their Appendix B) stated: " Contact surfaces of joints where slippage into bearing of the bolts is acceptable, may carry a shop coat of protective paint or lacquer; but in joints which will be subject to stress reversal, impact or vibration, the contact surfaces must not be painted or lacquered." Mr. Husband asked if we could assume that, because of the load on the bolt, those unpainted contact surfaces would not rust. It was quite uneconomic, he said, to have them perfectly machined and, if they were, it was quite likely that the coefficient of friction would be reduced and we should lose efficiency. On the other hand, he assumed that part of the success of that type of joint was due to slight irregularities on the surface, which tended to increase the friction. But those irregularities would let in the atmosphere, and he asked if we were completely right in not having a method of preserving the steel from corrosion; he would like to hear more about " glueing ". He did not like the idea of omitting all treatment of the contacting surfaces.

In conclusion, he said he did feel that more practical research of the type the authors had described was highly desirable, and he added his thanks to those already expressed to the authors.

MR. J. S. TERRINGTON (Associate-Member) commenting on the author's recommendation of the " half-torque half-turn " technique, asked whether, in view of the effects of scatter and since the effect of half torque was initially determined by the torque-wrench, this would necessarily give more accurate results, than either completing the tension by the wrench or by using a full turn, the effect of which was also known from the results of tests. In other words is a basic tension which relies on the torque-wrench before the application of the final torque, sounder than other methods?

Judging by the large attendance on that occasion it looked as though the need for the use of high-strength bolts was out-stripping fundamental knowledge of exactly how they should be installed in practice to derive the best advantage and achieve the maximum factor of safety. He quoted an example of a firm of crane makers, a group of engineers who were inclined to be highly conservative, who even contemplated the extensive use of high strength bolts. He would therefore like to thank the authors for their paper which had, in effect, been probably the first to initiate discussion on such an important subject in Great Britain following the wide applications already made in America as well as those in this country described during the meeting.

DR. ROYSTON JONES (Associate-Member) expressed his thanks for the opportunity to speak on the subject.

The notes he would give would be on behalf of Mr. P. Prynne, B.Sc., (his research assistant) and himself.

At Leeds University investigations on the behaviour of high tensile bolted connections had been concerned primarily with bolts stressed below the yield stress. Amongst other things the results illustrated the practical difficulties that site engineers might have in ensuring closely defined preloads in bolts. Indeed, these difficulties suggested that it might have been of more immediate use to the site engineers if greater attention had been given to joints with bolts at stresses at or beyond the " yield stress ". The Americans had shown little concern about exceeding the yield or proof stress; but in this country and in Germany it had been generally assumed that bolt stresses above yield were undesirable.

If bolt stresses above yield were permitted, the problem of joint assembly was simpler, and the " half-torque half-turn " method of tightening bolts seemed a sound suggestion, provided that the bolts had a reliable minimum yield stress and an adequate range of strain between yield and failing stress. Nevertheless, there was still room for further investigations of joints in which bolts were subject to both external shearing and tensile forces. The bracket joint shown in Fig. 4 of the original paper might be taken as an example. It had been written in the paper that from elementary theory, it would appear that, provided the preload in the bolts was sufficient to prevent separation of the plates, the effect of bending would have no influence on the shear carrying capacity of the joint. But there could be joints of this type which might have to resist a load varying from a high bending moment to a high shear load. Under a high bending moment—maybe even an accidental high bending load during erection—it could be assumed, for approximate analysis, that the bending moment would have the effect of applying a direct tensile load T to the upper bolt (or bolts). A consideration of equation (6) would show that the increase of load in the bolt would be $\dfrac{T}{1 + \dfrac{k}{K}}$; if the bolt were initially at the yield stress, the effective stiffness of the bolt K would not be based on the initial E, but on a tangent E obtained from

appropriate points on the stress/strain curves ; thus on release of load there would not be a full recovery of preload. That loss of load could be expressed in the form :

Loss of preload

$$= T\left(\frac{1}{A_pE_p + A_BE'_B} - \frac{1}{A_pE_p + A_BE_B}\right) \times A_pE_p$$

where A_p and A_B were the effective areas of the plate and bolt respectively, E_p and E_B were the moduli of elasticity and E'_B was the tangent modulus of the bolt at yield stress. Calculations made from data given for an American bolt of 1 in. diameter, tightened to its proof load of 29 tons, showed that a 15 ton direct tensile load applied to the bolt would cause a loss of 9.4 per cent of the original preload. The effective area of plate compressed had been assumed to be about equal to the area under the washer ; measurements of pressure distribution between clamped plates suggested that that was fairly near the truth. This showed at least the possibility of some reduction of the resistance of the joint to slip when subjected to subsequent shear loads. The American engineer Munse remarked that some have mistakenly reasoned that the tension in tension fasteners increases immediately upon application of an external tensile load, but later he added that stresses were not additive until the applied load had been increased to a value equal to approximately 0.6 of the initial tension.

Many arguments might be made in favour of the reliability of joints with bolts at yield stress ; the proof of successful practical application must also be accepted; but it was suggested that a fairly wide range of tests was necessary to prove the reliability of different forms of joint.

The tests at Leeds would be reported fully in due course, but in view of the remarks about different methods of tightening bolts, some diagrams had been chosen showing the variations in tension developed in bolts tightened on to a cylinder using three different methods—torque, turn of the nut, and part torque and 83° turn method. The bolts were $\frac{7}{8}$ in. diameter and all complied with B.S. 1083.

Average percentage variations in bolt tensions

Bolt Type	*Torque Method*	*Angle Method*	100 *lb.ft. torque and* 83° *turn*
4 BSW	29.4	28	17
6 Spec.	32.6	46	43.6
7 BSF	20.6	32	26.4

The graphs of Figs. 24 and 25 showed the relation of torque to tension ; Figs. 26 and 27 showed the relation of angle of turn to tension. Also shown on Fig. 27 was

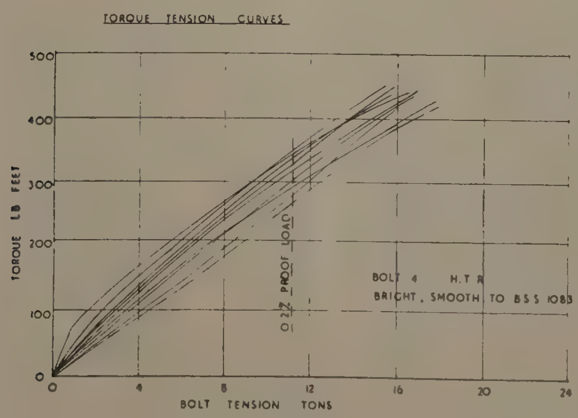


Fig. 24

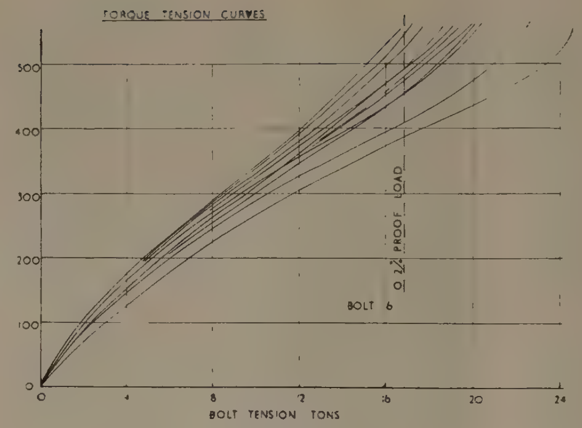


Fig. 25

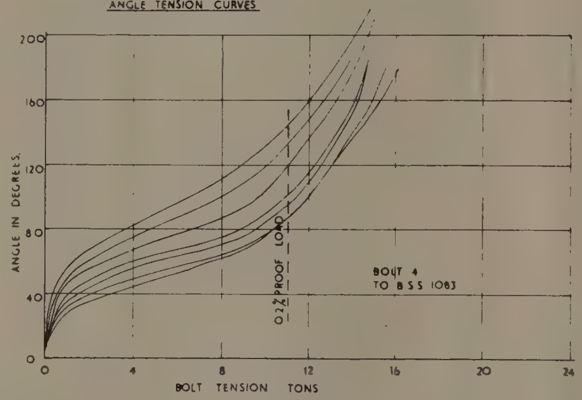


Fig. 26

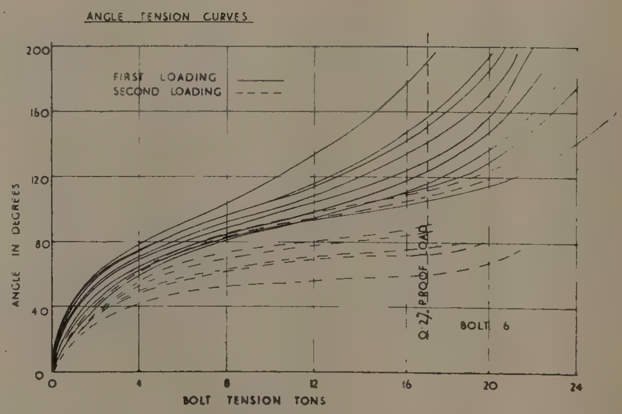


Fig. 27

the effect of releasing and retightening bolts. These results showed that for the type of bolts tested there was no very special advantage in the use of the part-torque part-turn of the nut method of tightening, if the object were to obtain a defined preload below the yield stress. If yield stress was desired, he thought the part-torque part-turn method was the right one.

Some reference had been made to joint rigidity. It was as well to note that there was some movement as soon as load was applied to the joint. During the

tests at Leeds the observed relative movement before the major slip took place was only about three thousandths of an inch. But conditions could be much more rigorous if, for example, rotational forces were involved, as might occur in a shear connector. In simple joints, rotational forces had reduced the slip load. If, in structural design, full advantage were to be taken of the increased joint rigidity given by high tensile bolted connections compared with similar riveted connections, more exact information must be obtained by tests of the appropriate types of joint.

Dr. Royston Jones added that there had been very little mention of the values for the coefficient of friction. The thing that puzzled him was the variation of coefficient of friction with sandblasting and flame cleaning obtained by the Germans who had reported something like 0.57 as an average of their coefficients of friction, compared with something between 0.3 and 0.4 with the normal plate. In their researches at Leeds he and his colleagues had not been able to get 0.57, with flame cleaned plates. Metallurgists had been consulted about the different characteristics of the German and British steels and so far no reasonable explanation had been obtained. Finally, it occurred to him that, as bits and pieces of work were going on throughout the country, it would be of advantage if at some time or other those concerned with the work could have some short round-the-table "clearing house".

Mr. W. C. Brown, after adding his congratulations to the authors for their most opportune paper, said that, as stated in the paper, when friction bolts were introduced to structural engineers by the American bolt manufacturers they were intended to be used as a direct replacement for mild steel field rivets.

That was unfortunate, for although it provided simple rules for the design of joints, it failed to give a true picture of the action of the bolted joint and, what was more, it did not provide the same load factor against slip, as did the rivet against shear.

In the paper the authors had stated that that load factor was 1.6, but a $\frac{7}{8}$ in. bolt, tensioned to 14^{T} and replacing a mild steel $\frac{15}{16}$ in. rivet, could slip at 4.1^{T}, whereas the rivet working load (based on a shear stress of 5.0 tons/sq.in.) was 3.5, i.e., a load factor of 1.2 only was obtained.

There might be some justification for that reduced factor in structures under static loading when slip failure was not catastrophic ; in fact, German tentative grip bolt specifications acknowledged the fact and permitted lower load factors.

When wind and reversal loads had to be carried, this factor seemed to be too low and would be further reduced by repeated slipping of the plates. Thus, either more or better bolts must be used.

Expressing his interest in the authors' comments on the torque measurement method of assessing the bolt load, Mr. Brown said that whereas the pneumatic wrench was a very convenient way of applying torque, the normal machine was very inaccurate in measuring the torque applied. Special machines existed which gave more accurate control, but all had to be calibrated and were dependent on air pressure to some extent for their efficiency.

Other types of bolts, such as the Torshear type, that were being developed, did much to eliminate the variation in the torque applied ; but it was the load on the bolt that we wanted to know, the torque was of little consequence. It was there that the turn of the nut method proved its advantages.

There were, however, disadvantages with that method when used with straight shank bolts and ordinary hex nuts. Unless the threaded portion of the shank extended between the plates, the bolt shank stretch was limited and there seemed to be a danger of the bolts breaking or, worse still, partly breaking off and remaining in the joint undetected. Also, with the standard form of small hex nut there was a possibility of the nut swelling at the base, seizing, and tearing the bolt threads.

One way of overcoming those difficulties was by using a reduced shank bolt with a special nut and washer, as developed by Mr. G. Roberts. The bolt was more or less a standard high quality waisted bolt, as used in the automobile industry, but the nut was turned down for $\frac{5}{16}$ in. and the shoulder bore on a thick high tensile steel washer so that the load was applied to the nut near the centre of its length. This reduced the concentration of stress on the end threads and prevented seizing.

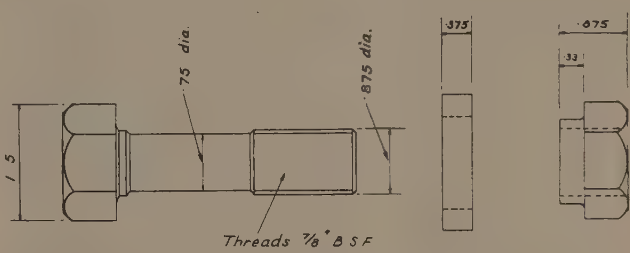


Fig. 28

With that nut and washer the failure in tension always occurred in the shank and the bolt could only be broken by three or four turns of the nut after hand tightening. Three quarters of a turn, after bedding down, resulted in a bolt stress near the ultimate tensile stress of the steel, i.e., the maximum capacity of the bolt was obtained in perfect safety. A bolt load of up to 35 tons might be obtained in that way with considerable accuracy by using a steel of 65/75 tons per sq.in. and an impact wrench capable of applying a torque at least 1,000 lbs. ft. Plates connected with one such bolt would slip at between 11 and 12 tons, compared with about 4 tons for the same size parallel shank bolt in En15R quality steel. Surely the economy lay in favour of the waisted high quality bolt.

Mr. Brown asked if the authors had any experience of surfaces which were sand blasted or flame cleaned, and whether they could comment on the economics of such processes.

Finally, he said that if engineers had confidence in the ability of bolts to carry loads effectively in tension, that would seem to be the best way of connecting units, particularly cross girders, to trusses, when at least one third of the number of bolts used in connecting the units in shear could be saved. The Germans used plain machined bolts rather widely.

Mr. M. F. Palmer (Member) asked the authors whether there were any criteria for the area of surface in contact per bolt. In practice, one met cases where (say) a plate was welded to the end of a joist section with welds around the profile of the section ; although for practical purposes the plane of the plate could be kept fairly true, yet the welding would have produced

some distortion of the end plate. Through the end plate bolts were placed and tightened but it was almost impossible to bring the whole area of the plate into contact with its meeting surface. He asked if the authors had carried out any experiments to ascertain whether there was any relationship.

Carrying the point a stage further, he asked if there was any relationship between plate thicknesses and bolt tensions in a lapped joint. He had in mind plates 2 in. or 3 in. in thickness.

Referring to Fig. 1 in the paper, showing a bolt through three thicknesses of plate, Mr. Palmer said it showed that friction took place over a fair area. If that was a joint with two cover plates, some of the friction would be taken up before the hole in the bar was reached. He asked whether any of the tests that had been made indicated that in fact the bar was stronger on this account. The thickness of the covers was involved, of course, but it seemed that the principle might apply.

Reinforcing the comment that torque multiplying spanners were slow in action, he said that was a very practical point, and it seemed to him that the future lay more in the line of the power operated impact spanner. Furthermore, if one had a number of bolts to tighten and if one was able to use a long lever, applying a torque of (say) 400 lb.ft. at an arm of about 4 ft., it was really the limit of a man's capacity and would be very tiring work when carried out continuously.

THE PRESIDENT, whilst realising that a number of others would like to contribute further to the discussion, said the authors should be given some time in which to reply. He therefore closed the oral discussion and invited written contributions.

Written Discussion

MR. R. W. TURNER (Associate-Member) invited the Authors' comments on the following points :

(1) The proposal to reduce the preload in H.S. bolts because of thread stripping appeared to be a retrogressive one. Would the adoption of bolts made to ASTM A 325 with UNC threads overcome this problem? Bolts to this specification were manufactured in England and their use with the highest permissible preload should give maximum efficiency of joints.

(2) Could the authors explain fully the equipment they proposed should be used to induce the half-torque and half-turn stages ? Mr. Turner had found the use of torque multiplying hand operated spanners inefficient and he considered that to use two different impact wrenches, one light and one heavy capacity would be the most satisfactory solution to this question.

(3) The proposal to use Torshear bolts which break off when a limiting value is reached was questionable because of the interaction when tightening a multi bolt joint. It had been frequently observed that one bolt being tightened caused adjacent (previously tightened) bolts to loosen but with Torshear bolts it was impossible to re-tighten and correct this tendency.

In the two stage method proposed by the authors the initial half-torque stage did have the effect of " bedding down " the plies and was extremely valuable because of this action.

(4) As the condition of the faying surfaces greatly affects the frictional resistance of a preloaded joint had the author any proposals for calculating the capacity of joints based upon the grade of surface finish, i.e. to decide upon average values of " μ " for " flame cleaned " " grit blasted " " painted " and " as rolled " surfaces ?

MR. WILLIAM H. ARCH felt that the references to the turn-of-the-nut methods of tightening made through the paper, and the verbal discussion, did not make clear what type of thread was used on the bolts the use of which was being described. If any " turn " method of tightening was to be stipulated it was abundantly clear that the thread intended should also be given. There was a large variation between the tension to be expected from half a turn with a B.S.F., B.S.W., or the equivalent unified threads. The U.N.C. thread had been found to be most useful in this field, and was far less prone to thread stripping than was a fine thread.

There was also the point that, strange though it may seem, there was far from being a constant relationship between the amount of turn applied to a nut and the tension resulting in the bolt therefrom.

MR. J. B. GRIFFITH considered that the substitution of high preload bolts for rivets was very attractive, particularly in view of the fact that riveters today are in such short supply. Moreover, site erection of steel structures would clearly be made much simpler if the authors' recommendations were to be generally accepted and adopted.

Whilst he had not had any experience in the field covered by the Authors in their paper, Mr. Griffith had met problems when post-tensioning pre-cast concrete units which he felt might have some bearing on the important problem of the use of high preload bolts. In the particular case he had in mind, $1\frac{1}{8}$ in. diameter Macalloy bars were used for post-tensioning pre-cast concrete cylinder units to form cylinders up to 65 ft. in height. The bars were tensioned vertically ; the lower end being anchored by bearing plate and nut in a cast steel shoe, whilst the bar was then jacked at the upper end. A standard Lee-McCall prestressing jack was used, the jack being attached to the bar by an adaptor. Measurement of tension (which was fairly critical), was carried out by two methods :

(a) Suitable calibration of a Bourdon gauge coupled into the jack's hydraulic system.

(b) Measurement of extension of the rod, from the time when the rod begins to take load.

There was nothing unusual about this system. However, they did carry out experiments to determine the effects of slight misalignment of the bearing plates at top or bottom of the bar. Whilst the average ultimate strength of the bars held in bearing plates square to the test bar was over 63 tons, failure occurred at under 50 tons when the plates were set between $1\frac{1}{2}°$ and 3° out of true. Using *short* test lengths with spherical faced nuts and spherical faced washers to permit nuts and rods to align themselves, the effects of misalignment were greatly reduced, though due to waviness of the bars and friction in the self-aligning bearing these results did not hold good for the anchored end on *long* bars.

The reason for the reduction in ultimate strength in the above tests was obvious but he wished to ask the Authors' opinion on whether they felt that some of the failures in their earlier tests may not have been due to lack of appreciation of the effect of slight misalignment. Dr. Wells' suggestion that a lead alloy ring be used to gauge tension might also solve this problem.

Secondly, the half-torque half-turn technique developed by the Authors was, like all good answers, simple. The problem, however, was the determination of the point at which axial load commences to be applied, particularly where there were more than two plies of material. The Bourdon gauge method described above was, within the very doubtful limits of accuracy of the gauge, a true measure of tension. The nut could then be run down and the jack released. Whilst lack of room would usually preclude the use of such a method, did the Authors not agree that a fundamental approach was required rather than the placing of reliance of torque limiting spanners, personal judgment of the point at which the bolt commenced to take tension and allowance for springiness of the material, particularly where multi-ply bolting was used? Approach along the lines of Dr. Wells' suggestion seemed likely to overcome these problems.

Mr. R. Newsham invited the Authors' comments on the following observations and questions.

A term which frequently occurred in the text of the article was "bolt yield point" and he wondered exactly what was meant by this expression. Did they in fact refer to the deformation which occurred in a nut and bolt thread assembly and if so how did they assess when this had occurred. Further would they say how the loads in the bolts were measured in their tests also how many bolts and what type were used.

From the tests which Mr. Newsham had made the factor leading to the greatest variation appeared to be the method of forming the threads, a considerably higher scatter was obtained when the threads were "cut" than when they were produced by a cold rolling process. That might be an important factor in the correct assessment of the preload in H.S. bolts and it was considered that in attempting to produce a "standard" to which the bolts and nuts should be made careful thought should be given to the methods of production and to the tolerance to be allowed during manufacture.

When considering the very high stresses which occurred in the first few threads of a nut and bolt assembly it seemed remarkable that such an improvement was obtained in the torque-tension characteristics when the threads were treated with tallow as opposed to those tested in a dry condition. This would indicate that tallow itself was able to withstand very high bearing pressures. In Mr. Newsham's tests even a graphite-grease compound did not produce any consistent improvement in the results.

With regard to the value of the coefficient of friction which was generally used for bolted joints a value of as high as 0.55 was obtained with a high strength bolted friction joint which had been correctly fitted. It was considered that this value would have been even higher if a higher clamping force had been used.

It would appear that whilst high strength bolts may be used in a "friction" joint with safety, care must be taken when they were used in a "tensile" joint where it was much more important to ensure that the mating surfaces should be "fitted", that is there should be no gaps between these surfaces, around the areas of the bolt holes, as these might produce stress variation which would in turn lead to fatigue failures in the bolts.

Reference

* Erker, Dr. A., "Design of screw fastenings subject to repeated stresses," *International Conference on Fatigue of Metals, Inst. Mech. E.* September, 1956.

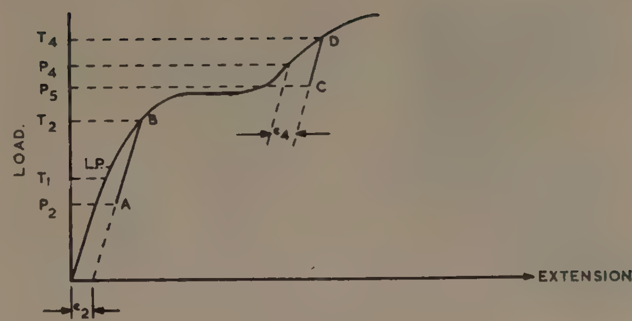


Fig. 29

Mr. T. R. Gurney commented that while much of the authors' paper had been devoted to shear connections it must not be overlooked that in a large proportion of these so called shear joints the bolts carried applied tension loading as well as shear. This might be due to an actual tensile load as shown in Fig. 3, or due to a bending moment as shown in Fig. 4, or to a combination of the two. Although there would appear to be good reason to expect satisfactory service from preloaded bolts in pure shear connections, various experimental results showed that the same could not unfortunately, be said without reservation of bolts which may suffer any applied tensile loading.

A bolt having a load extension curve as shown in Fig. 29 was considered as an example. Theoretically, if the actual load in the bolt T_1 (= the preload P_1 + a proportion of the external applied load R_1, which depended on the stiffness ratio of the bolt and abutments) did not exceed the limit of proportionality (L.P.) of the bolt material, the behaviour of the bolt would be entirely elastic and preload would be maintained. If, however, under an applied load R_2 the actual load in the bolt was T_2 (greater than LP but less than yield point) the bolt would suffer a permanent set e_2 and the clamping load would fall to a new value P_2. On further applications of the same load R_2 the bolt should load and unload along the line AB, although any increase in the applied load R would result in further permanent set and more loss of preload. In particular, if the load in the bolt rea hed a value $T_3 \geq$ yield load, a large permanent set would result leading to complete loss of preload. Since this yielding was partially time dependent it might require a number of short duration load applications for this complete loss to become apparent.

But, if the initial preload were taken to a value P_4 in the strain hardening range, an applied load R_4 would result in an actual bolt load T_4 and there would be a permanent set e_4 from the preload condition leading to a drop of clamping load to P_5. On further applications of the load R_4 the bolt might be expected to behave elastically along the line CD with no further loss of preload, but according to results given by Dr. Erker* this was not correct and there was a steady loss of clamping load on repeated applications of the load R_4.

Mr. Gurney had carried out a limited number of tests on $\frac{3}{4}$ in. diameter mild steel bolt studs, with the shanks turned down to the root diameter. The material had a limit of proportionality of approximately 16 T/in^2 and a lower yield of approximately 23 T/in^2. The experimental set-up was outlined in Fig. 30. The studs were preloaded by tightening the nuts and external loads were then applied. After 10 applications (each for 15 seconds) of each load the

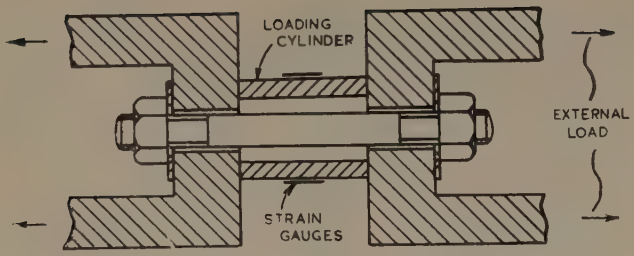


Fig. 30

remaining clamping load was measured by electrical resistance strain gauges on the cylinder, and it will be seen from Fig. 31 that as the applied load was progressively increased the remaining clamping load after each series of 10 applications decreased until, after applying the yield load, all preload was lost. Similar results had been quoted by Dr. Erker.

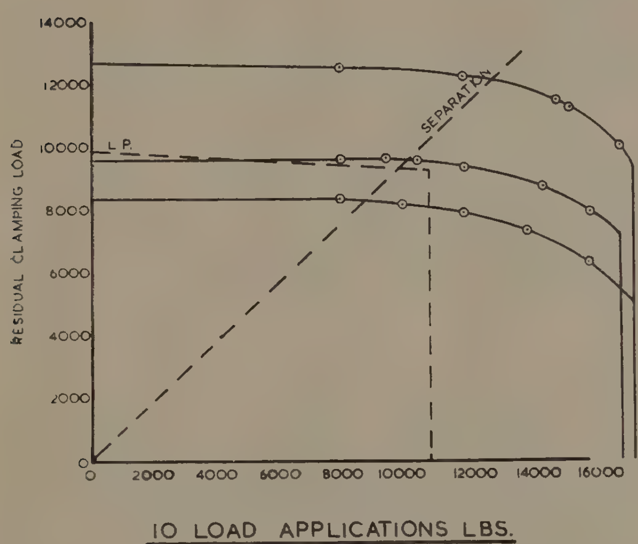


Fig. 31

It therefore appeared that, before preloaded bolts were generally used in connections where they might suffer extra tensile loading, some research should be carried out to define more clearly their behaviour. For the moment all that could be said with any degree of certainty was that, if preload were not to be lost, the actual stress in the bolt should not exceed the limit of proportionality of the material.

Authors' Replies to Questions

Since many aspects of the same subject had been raised by several contributors it seemed to the authors that the best procedure was to divide their reply by subject matter, referring to the points raised by the contributors and so avoiding the repetition which would result from individual replies.

Nomenclature

The authors were unable to agree with Mr. Berridge on the term " High Strength Bolt " and were unrepentant for the introduction of the term " High Preload Bolt ". Fundamentally the essential feature was the high preload and whether it was achieved by a small bolt highly stressed or by a larger bolt less highly stressed was irrelevant. It was understood, however, that in official circles the term " High Strength Friction Grip Bolt " was currently gaining vogue and the Authors would gladly withdraw their suggested terminology in its favour.

The authors apologised for not being explicit in the Paper when they used the term " bolts above yield " and were grateful to Mr. Newsham for pointing this out. By the term " yield load " the authors meant a load sufficient to raise the stress in the " effective stress area " of the threads to the 0.2 per cent proof stress of the material.

Half Torque Half Turn

A number of questions had been asked about the part torque part turn technique proposed by one of the authors. First it should be emphasised that the technique was not intended to induce a defined preload below the minimum guaranteed yield of the bolt but to induce with safety a preload at least equal to the actual yield of the bolt. The part torque was simply to ensure proper bedding down of the whole joint before the part turn was applied. This part torque was purely a nominal " half torque " and might be anything between one-quarter and three-quarters of the torque which would be normally applied to tighten the bolt using the torque coefficient technique. The reason for accepting such a large tolerance could be seen in Fig. 15 which was typical of the results obtained from tests on studs and bolts of different length and diameter. Examination of the limiting torque curves showed that the final preload after the part turn (in that case three quarters turn) was only slightly affected by the amount of torque applied in the initial bedding down. Crude and relatively inaccurate devices could therefore be used to apply the part torque. Indeed a torque spanner might be considered unnecessary and the bolts tightened " as tight as possible " using an ordinary erector's spanner. The part turn was added only after the part torque had been applied to all the bolts in the joint. A paint mark on the nut and protruding threads of the bolt would assist the erector in judging the part turn correctly and also permit the actual turns of the nut to be subsequently checked by an inspector. The part torque part turn technique had been applied successfully with $\frac{3}{4}$ in. to 1 in. diameter bolts having B.S.W., B.S.F. and U.N.C. threads and, although they had no experience of that type, the authors saw no reason to think it invalid in the case of U.N.F. threads. Due to variation in the effective stress area of the threads there was some variation in the preload for bolts of the same diameter, but in all cases the preload was greater than the actual yield load. It seemed to the authors that a single tool might be used for applying the technique—an impact wrench capable of applying the final part turn could easily be modified to indicate the part torque. This answered points raised by Mr. Terrington, Mr. Arch and Mr. Turner.

Bolts Beyond Yield

Up to that time the authors had only a limited service experience of bolts tightened beyond yield but American experience over a number of years had found bolts so tightened to be successful. Mr. Creasy mentioned that when impact wrenches had been used in several successful contracts there had been a small

number of bolt failures ; it might therefore be deduced that a number of the remaining bolts were tightened beyond yield. In passing it might be mentioned that the authors were currently carrying out fatigue and strain-ageing tests on bolts tightened beyond yield.

Mr. Brown expressed the fear that since " turn of the nut " techniques developed preloads at least equal to the yield such techniques might be dangerous when used with straight shank bolts and ordinary hexagon nuts. Over fifty tests to failure with straight shank and waisted bolts in En.110U, En.16R and B.7a quality steels had been carried out by the authors and there was no evidence of partial cracking of the bolts prior to failure. It was thought that with these steels and high working stresses the chances of arresting a crack once started would be slight. As a result of early tests, however, it was found desirable with En.110U and B.7a steel bolts (0.2 per cent proof stress 48 tons per sq. in.) to use a chambered nut to prevent belling. The " Roberts nut " described by Mr. Brown has a similar effect plus the additional advantage of spreading the load along the threads. This " evening out " of the load had the same effect as waisting the bolt in that both relieved the tendency for failure to take place about the first or second thread inside the nut. The use of a " Roberts nut " with a waisted bolt might perhaps be unnecessary.

With regard to the nut troubles discussed in the Paper, one of the authors commented that he had experienced in one contract a 2 per cent failure rate due to stripping of the threads. In reply to Mr. Turner the authors pointed out that the preload was not reduced because of thread stripping. With the thread finish and lubrication particular to this contract the torque coefficient was found to be about 20 per cent lower than that which had been supposed, in other words, the preload induced by the applied torque was higher than was expected. Thread stripping was due to this excessive preload.

Joints Loaded in Shear

The effect of surface treatment on the coefficient of friction, or slip coefficient as it was perhaps better termed, was raised by several contributors. The authors had not yet carried out tests of this kind but Dr. Royston Jones and Mr. Newsham reported the results of such tests. Mr. Creasy showed graphs which, on analysis, led to a slip coefficient of .45 for clean, dry surfaces. One of the authors carrying out tests in another field to determine the slip coefficient between a spring steel clip and a gantry rail observed slip values as high as 1.0. In view of this the authors felt that Mr. Newsham might well be right in suggesting that larger slip coefficients might be obtained, particularly if very high bolt preloads were used together with small washers. This remark was quite aside from such developments as the introduction of glue-like materials between the plies.

This led to Mr. Palmer's question concerning the effective contact area between the plies. The authors could do no better than refer him to Dr. Royston Jones' contribution in which he reported that the contact area approximates to that of the washer.

When discussing joint rigidity under load, Dr. Royston Jones reported that the relative movement of the plies which occurred before major slip was less than .003 in. whilst analysis of Mr. Creasy's graph shows his slip to lie in the range .004 in. to .011 in. The authors suggested that perhaps these anomalies arose from different experimental techniques. Possibly Mr. Creasy's results included some rotation of the joint, as would be the case if the deflections were measured over the whole length of the specimen.

Joints Loaded in Shear and Tension

Dr. Royston Jones and Mr. Gurney raised the question as to the behaviour of brackets carrying shear and bending, so subjecting several of the bolts to both shear and tensile loads. At the time of writing the Paper the authors' proposal was to use the part torque part turn technique for joints carrying shear load only but, in introducing the Paper, they discussed development work to be carried out on joints subjected to various types of loading including tension. They had considered the particular case of the bracket type of joint and felt that the proposed technique was still suitable.

They would illustrate their point by considering the behaviour of the bolts under tension in such a joint. If the part torque part turn technique had been used to tighten the bolts the reduced preload when the external load was removed would be, taking the example quoted by Dr. Royston Jones, approximately .90 times the actual yield load of the bolt. If the torque coefficient method of tightening had been used then the initial preload development in the bolts before the application of any external load would be only .85 to 1.00 times the minimum guaranteed yield load. Since the actual yield was on average 10 to 20 per cent greater than the guaranteed yield the reduced preload after yielding of the bolt would still be greater than the initial preload developed by the torque coefficient technique. The percentage reduction in preload was, of course, a function of the external load but since it was customary to design the joint on a factor of safety of two against separation, a value of 10 per cent might be regarded as typical. Thus whilst the condition of the yielded bolts in the bracket would be no worse than if they had been tightened on the torque coefficient technique, the remaining bolts would not suffer yielding and would retain their high preload. A bracket tightened up using the part torque part turn technique would, even after yielding of several bolts, still have a greater total preload and hence a greater factor of safety against slip. The authors would, however, endorse the point that cognisance must be taken in design of this loss in preload.

While the foregoing covered what the authors believed to be Mr. Gurney's main point, they wished to comment upon a further point. Even if the stress in a bolt was not allowed to exceed the limit of proportionality there might still be some loss of preload on cyclic load application. If there was insufficient preload to prevent slip and/or separation of the plies, fretting between the faying surfaces could cause a continuous loss in preload.

The Design of Joints

While the authors agreed with Mr. Husband that basically the bolted joint served the same purpose in structural work as a riveted joint, they deprecated any suggestion that the designer should think about the bolts as being " replacements " for rivets. They suggested that such thinking neither significantly simplified analysis nor led to the best design.

At this juncture they felt it might be helpful to indicate the design method used by one of the authors. Using the part torque part turn technique, the preload in each bolt was assumed to be equal to the minimum guaranteed yield load. Any joint subject to cyclic

shear, bending and/or tension must satisfy the following relationships :

(a) $n W_p \geq 1.5 S/\mu + 2.0 P$

(b) $p \leq W_{p/2}$

where W_p = preload in each bolt.

n = number of bolts in the joint.

μ = slip coefficient, assumed equal to 0.4.

S = total external shear load on joint.

P = total external tensile load on joint.

p = the maximum tensile load on any bolt in the joint.

When discussing American practice it was stated in the Paper that if a rivet was replaced by a bolt of the same nominal diameter the factor of safety against slip was about 1.6. In reply to Mr. Brown the authors stated that the shear capacity of the rivet was calculated on the diameter of the rivet and not on the diameter of the hole. It would also be seen from Table 4 of the Paper that the proof stress of the bolt varied from 38 tons per square inch for a $\frac{5}{8}$ in. diameter bolt to 33 tons per square inch for a $1\frac{1}{4}$ in. diameter bolt ; an average proof stress of 36 tons per square inch was therefore used.

Torshear Bolts

The authors agreed with Mr. Berridge as to the simplicity and neatness of the Torshear bolt. Here an attempt had been made to induce a preload which was related to the actual proof stress of the bolt, as the torque applied to the nut was a function of the shear resistance of the grooved shank of the bolt. Since the preload was, however, dependent upon thread finish and lubrication the authors suggested that, even for bolts of the same proof stress, the preload might still be subject to considerable scatter. The authors endorsed the point made by Mr. Turner that, due to the loosening of previously tightened bolts the complete tightening of one bolt at a time was a poor way of tightening a group of bolts. The Torshear bolt could be first partially tightened to bed the joint down but it must be borne in mind that once the bolt had sheared the elegant power tool could not be used for further tightening.

Future Development

In conclusion the authors endorsed most wholeheartedly the suggestion by Dr. Royston Jones that a symposium be arranged for interested parties to discuss the future development of this new technique. Such a symposium might possibly be organised under the aegis of the Institution and would seem both timely and extremely valuable in this year of the 50th Anniversary of the Institution.

Book Reviews

Dynamic Instability by Y. Rocard. (London : Crosby Lockwood, 1957). $8\frac{1}{2}$ in. $\times$ $5\frac{1}{2}$ in., 227 plus xi pp. Price 45s.

This book contains a detailed mathematical treatment of certain dynamic instability problems in road vehicles, aircraft and suspension bridges. Structural engineers will naturally be most interested in the last mentioned problem to which 94 of the 218 pages of the text are devoted.

The spectre of the Tacoma Narrows Bridge must still loom large in the minds of those responsible for the design of large suspension bridges and to them Professor Rocard's lucid and comprehensive analysis will be of great interest and importance.

In the opening chapters the author discusses the underlying theory of oscillations, including coupled oscillations which are of particular importance in relation to suspension bridges. In the section on suspension bridges the various modes of oscillation are examined in relation to the aerodynamic effects which are likely to excite them. Certain of the theoretical results are applied to the Tacoma Narrows Bridge and in general the observed behaviour of this bridge confirms the expectations of the theory. The effects of structural damping and of tower deflections are also examined. There is a brief review of American work on the subject but there is no reference to the considerable amount of theoretical and experimental work which has been carried out in recent years by Selberg, Frazer, Scruton and Pugsley on this side of the Atlantic ; the author, however, does not claim completeness for his bibliography.

The book has been translated from the original French by Mr. M. L. Meyer of the University of Sheffield who has succeeded admirably in preserving the author's clarity of presentation.

Laboratory Testing in Soil Engineering by T. N. W. Akroyd. (London : Soil Mechanics Ltd., 1957). 10 in. $\times$ $7\frac{1}{2}$ in., 233 plus xx pp. Illustrated. Price 35s.

This book describes a procedure for testing the engineering properties of soils evolved in the laboratories of the firm of Soil Mechanics Ltd. over a number of years, and incorporates many ideas from other laboratories and from workers in other countries. A very comprehensive series of tests is described, comprising moisture content tests, index tests, sizing analysis, comprehensive shear tests, direct shear box tests, consolidation tests, permeability tests, compaction tests, the California bearing ratio test, and soil stabilisation tests.

The book is well produced on good paper and with clear type and illustrated by thirty-two plates and thirty diagrams. The apparatus required for the various tests is listed in an appendix, and a bibliography concludes the volume.

The Problem of Mechanical Handling in Building Operations*

Discussion on the Paper by J. F. Eden, B.Sc.(Eng.), M.I.Mech.E., and D. Bishop, A.M.I.C.E., A.R.I.C.S.

THE PRESIDENT, introducing the authors, said that the paper was the result of the combined efforts of a mechanical and a civil engineer. Their work illustrated the valuable way in which the Building Research Station, like so many research establishments, had extended its operations since the war.

The meeting looked forward very much to the presentation of the paper, and to a film which illustrated mechanical handling in building operations.

MR. EDEN presented the paper, which, he said, was not a research paper, but concerned the selection of plant for mechanical handling.

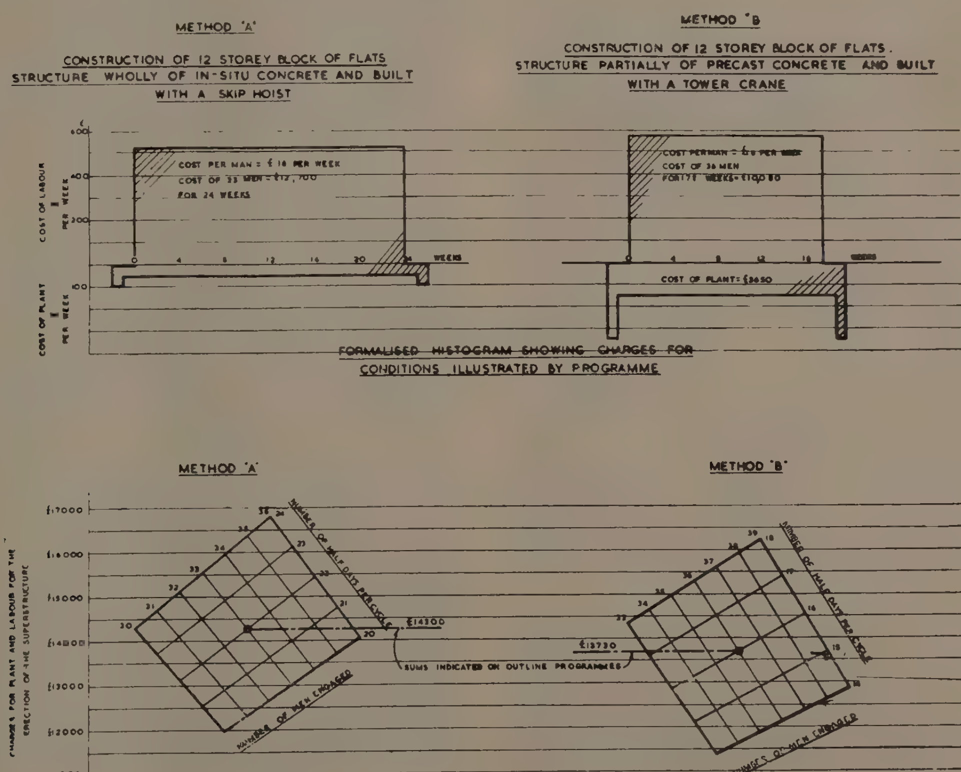


Fig. 7

He illustrated it by means of his film and a series of lantern slides. Mr. Eden emphasised that the effect which handling plant could have on reducing costs could not be disclosed by any direct comparison of handling costs, and that estimates of the cost of the whole job, prepared from preliminary programmes of the type shown in Tables I and II were necessary. Bearing in mind however the inexact character of such estimates, some appreciation of the effect of likely errors was advisable before a decision was taken. In this respect, other representations were useful. Thus the histograms at the top of Fig. 7 showed the same estimates, giving the day by day expenditure on labour and plant. These disclosed the relationship between plant and labour costs and the shape was informative. Thus the total plant and labour costs, as depicted by the area between the two curves, was more seriously affected by the additional men on the site than by the job taking a few days longer, if the histogram was long and shallow, whereas a shorter, squatter histogram as method B was more sensitive to variation in duration of the contract than to other factors.

In both these examples it was clear that labour costs predominated so that a useful question was " How far am I likely to be out in my estimates of the

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1., *on the* 27*th February*, 1958. *Professor Sir Alfred Pugsley, O.B.E., D.Sc., F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S., (President) in the Chair. Published in " The Structural Engineer," Vol. XXXVI, No.* 2, *pp.* 61-72. (*February* 1958)

number of men on the site, and the length of the cycle time, in each case ?"

If limits of inaccuracy could be estimated, the effect of variations within these limits could be shown by drawing charts as described by McIntish in the Magazine of Concrete Research for December 1949. Comparable " Carpet Graphs " of this type were shown under the histograms. These showed exactly what must be achieved in terms of numbers of men and duration of contract for one method to show an advantage over the other, and also, in practical terms, of number of men and number of days, the latitude which could be allowed in either estimate. These pictures did not give answers, but helped a man to use his judgment to better effect.

Vote of Thanks

THE PRESIDENT expressed the thanks of the meeting to the authors for the most interesting way in which they had introduced their subject. Academic people, he said, were largely interested in design rather than construction, and the authors' presentation of their paper had been very educative.

Discussion

MR. E. U. BROADBENT first expressed his own appreciation of the authors' very good paper, an eminently practical one, at any rate from the point of view of the contractor. It gave a very fair statement of the difficulties facing the contractor in trying to evaluate the relative costs of different ways of doing things, and it gave proper emphasis to two things.

The first was the distinction between the manufacture and the assembly of parts of a building. Probably there was no industry which combined manufacture with assembly in the sense that the building industry did ; he did not know any other industry where the bits were assembled on the site to the same extent. Whilst quite clearly the time was a long way ahead when we could separate them completely, there were advances to be made in keeping manufacture off the site.

He preferred to treat with reserve some of the things he had seen in the film. It implied we could have bigger and bigger units and could get them cheaper and cheaper. When he had seen a gantry crane lowering a floor he had wondered on how large a scale contracts would have to be to make the scheme anything but wildly uneconomical.

He agreed we could not apply ordinary estimating methods to evaluating the savings effected in such work, but it was possible to go too far in the other direction. In applying considerations of the total time and the total value in the job, we must also give a great deal of thought to the amount of work that was involved before we could justify an investment on the scale of some of the plants illustrated by the authors.

Mr. Broadbent said he was fortunate enough to have had some little experience of the value of dissociating manufacture from assembly in the scheme at Picton Street, which was referred to in the paper, where his firm were able to do that to some extent. They precast all their floor units and balcony units and all the cladding panels, and because the designer was able to design the whole thing as an integrated operation, with a certain method of construction in mind, they were able to equalise the loads on the various items in the structure, with the result that they achieved very good utilisation of their plant, not only in terms of the number of lifts, but also of the number of tons lifted per hour. That was an important thing to remember.

As the authors had emphasised more than once, we were not dealing purely with the problem of how to erect a building on the site ; we were right back in the field of design, and it was from the first conception of the design that the ground was laid or the seeds were sown for successful mechanisation on the site. If the design fell down in that respect, no amount of effort on the part of the contractor who was erecting the building could lead to satisfactory results.

The remark had been made by Mr. Bishop, when describing the Sundh system, that Mr. Sundh had redrafted the design to suit the system of erection, but why did not the architect draft the design to suit Mr. Sundh's system ?

Finally, Mr. Broadbent asked if the authors could say whether calculations had been made as to the volume of work required to make the various systems economical, for it seemed to him that they had gone into very great detail in connection with very large schemes and had rather neglected the small schemes. We wanted means of doing a good job with the smaller schemes also.

MR. F. J. SAMUELY (Member) enlarging on one or two of the points mentioned, said that knowledge of mechanical handling was essential for the design of any structure, particularly where precast concrete was concerned.

A completely precast building could easily cost 10 per cent too much if mechanical handling was not properly thought out. It was absolutely necessary that the designer should know all about the possibilities of mechanical handling, and where it was possible it was very wise for him to consult the contractor beforehand. The cost of crane handling could in many ways be kept to a minimum. It was essential to avoid cranes standing idle for considerable periods, and proper design could help to reduce to a minimum the time necessary to place each part of the structure into position, so that the crane could be used immediately for the next unit. It was quite obvious that the maximum efficiency could be achieved if the plant were in use for the maximum time. He felt that, just as the designer put on to paper his ideas of what the structure should be, he should also indicate on paper how it was to be erected. Then, although one must allow the contractor to use a different system if he wished, there was one ready for him to adopt.

A great number of contractors had not adjusted themselves to mechanical handling. In his own experience there were cases in which cranes had been standing idle for 90 per cent of their time and where it was actually necessary to explain to the contractor that he had expensive and modern tackle which he must learn to handle. Therefore, Mr. Samuely pleaded that general knowledge about mechanical handling should be disseminated widely. Today we were only beginning to erect precast concrete structures, and there remained very much more to be done.

MR. C. D. MITCHELL, referring to illustrations shown in films, commented that the engineer's concept of a building was not always the architect's and we had got to bear in mind that what pleased the engineer would not necessarily please the architect ; the architect would have his ideas of prefabricated types of buildings. In this country we were not accustomed to the standardisation of buildings, we liked things to

differ and therefore when prefabricating we designed the larger pieces a little smaller than perhaps those featured in Continental contracts. This gave us great flexibility.

He considered that mechanical handling was for the contractor an extremely interesting and challenging problem. The introduction of the tower crane had revolutionised building in this and other countries and Mr. Eden and Mr. Bishop had pointed out that it gave rise to the need for a new kind of thinking. It meant that, if we wanted to achieve maximum efficiency, we could no longer indulge in the luxury of out of date tendering methods. The new ways of erecting multi-storey buildings made it necessary to plan the work properly and to call in a specialist contractor at an early date, preferably to play a major part in the design. Also, mechanisation was a great aid to the workmen, who were very pleased to work with the new methods. Those methods were very good from everyone's point of view and would undoubtedly lead to new forms of building.

He would like to congratulate the authors upon their paper and in particular for the excellent work in this field undertaken by the Building Research Station.

Mr. F. R. Bullen (Hon. Treasurer) confessed that when he began to read the paper he had the feeling that the prime object of erecting a building was to enable plant manufacturers to find a use for their plant; he had felt that the emphasis was on the good of the manufacturers rather than on the good of the man who was trying to erect the building. Also he had the feeling that the things that were said about designers were not quite as true as they might seem to be.

Nevertheless, as he continued to read the paper he felt that the authors had drawn attention to a most important point; it was vital that those whose job it was to design should know what plant was available and what methods there were by which buildings could be erected. So that, whilst he joined issue with the authors over the way in which they had presented the point, he agreed with them that it was necessary for designers to know more about construction methods.

Referring to the "utilisation factor" Mr. Bullen said he gathered that it meant the proportion of time during which the plant was usefully employed, as against the time it was standing on the site. That factor was a good deal less than 1, and the authors' purpose was to raise it as near to 1 as possible. It seemed to him that one aspect was the right choice of plant in the first place. It was no use saying that the design was bad if the choice of plant was bad, and he felt that there was something to be said for the education of contractors as well as of designers. There was a good deal to be said for contractors giving more thought to the choice of plant in the light of the design of the structures they were asked to build.

One of the photographs in the paper was of a job for which he was responsible, and he ventured to show a diagram.

He pointed to a circle of 60 ft. radius, which represented the radius of one of the Pingon tower cranes; it stood in the lift well of the block of flats, which was of "aeroplane" shape. The layout at this site was particularly good. He pointed to the positions of the crane and of the concrete mixer, the latter being fed by the sort of scraper referred to in the paper from the ballast heap, and also to the position of the cement silo, containing cement in bulk. There was one space

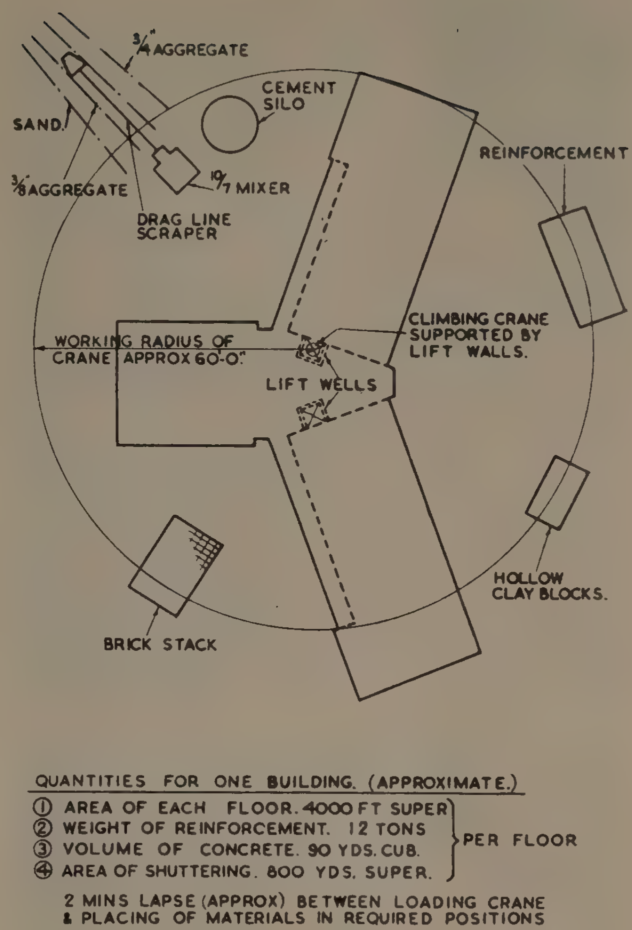


Fig. 8—Typical arrangement of Plant and Materials for continuous feeding of Crane.

for stacking bricks, another for hollow blocks and another for reinforcement. There were 10 floors above ground, the area of each being about 4,000 sq. ft., and the quantities of materials in each were approximately 12 tons of reinforcement, 90 cu. yds. of concrete and 100 sq. yds. of shuttering. The whole structure was cast in situ and the construction of each floor occupied exactly two weeks.

In that connection he referred to an office building which was constructed a year or two ago. It was very largely precast, covering about the same area as the block of flats, and the corresponding operation there occupied about two weeks. Again, at the Grosvenor House Hotel, where a combination of precast and cast in situ work was used, two weeks was the time required per floor. It seemed to follow, therefore, that, no matter how big or how small the area of a building might be whether precast or cast-in-situ, about two weeks were required per floor, say 10 working days and perhaps an extra day.

Mr. Bullen showed a photograph of one of the blocks of flats with another block in the background, and pointed to the crane on each. His purpose in showing this slide was to draw attention to a factor which might lead to improved production; it was possible to set one block against the other in regard to speed of construction, and indeed the work had been referred to as the Treverton Stakes. He suggested that there might be something in that psychological

factor in increasing the speed of construction and, therefore, improving the utilisation factor of the plant.

Another view was shown because it illustrated in the foreground one of the problems with which the structural engineer was faced. The structural engineer having very carefully arranged his details so that everything was repetitive from floor to floor, said Mr. Bullen, others required provision for large electrical conduits, some wanted large holes for various fittings, some wanted provision for rainwater pipes, and some wanted other things. All those requirements created difficulties which the structural engineer had to overcome and still retain the repetitive features.

The next illustration showed one of the frames very nearly finished. Mr. Bullen used the picture to draw attention to a point arising out of a comment by one speaker. The crane was supported within one of the lift shafts. He had wondered, when the speaker had said there was some advantage in precasting the lift shaft, how this could be done when using the lift shaft to carry the Pingon crane. Further the lift shaft was one of the main elements in taking up the wind stresses; so that if the design were changed to precast all the wind resistance would have to be put into the vertical columns, the size of which, presumably, would be much increased. Therefore it did seem possible to have both advantages.

Mr. R. A. Inglis (Associate-Member) said that, as a structural engineer, he wanted to rush to the assistance of architects, because in the mechanised production of buildings he felt we could under-stress the importance of the aesthetic considerations of design. In reality the appearance of things was very important to the general well being of people. He hoped that in the transition from the present relationships between design and production to those implied in the mechanisation of building construction such as had been so ably described that evening, aesthetic considerations would be borne properly in mind.

Secondly, the inference to be drawn from the processes described was that only by large scale production and continuity could the best results be gained from full mechanisation. We wanted to be able to produce quickly all the buildings we needed so urgently and therefore the "Stop-Go" processes which we had experienced since the war would prevent the proper realisation of such developments. It was most important that both Government and Local Authorities should appreciate the vital need for continuity in their building construction programmes.

Mr. W. Paszkowski (Associate-Member), speaking as a consulting engineer, made the following points:

1. The cost of reinforced concrete structure today was influenced mostly by the high cost of labour and shuttering—at least if the costs of materials like steel, aggregate and cement were reasonable. Therefore the majority of methods in use for the erection of precast concrete aimed to reduce the cost of labour and shuttering.

2. Mechanical handling in building operation reduced labour costs and time of construction, provided that design and site organisation incorporated every aspect of mechanical handling.

3. While the film shown illustrated the problem of mechanical handling, it did not present the best available practice in methods of construction of precast concrete.

4. Methods of precasting of reinforced concrete structure should not only simplify fabrication and erection but also provide good finishes, so that additional work such as plastering might be eliminated.

5. In some continental countries economy in construction was expressed in terms of a labour index and a shuttering index. The labour index represented the time taken for the erection of one unit of volume of building. For instance in Sweden 3 man-hours was used for the erection of 1 cubic yard of building.

The shuttering index expressed the consumption of timber in the erection of 1 cubic yard of building. In Sweden only 0.0015-0.01 of a cubic yard of timber was used for the erection of a cubic yard of building.

6. Several methods for the construction of precast concrete had been developed both here and abroad and perhaps the best of these at present was practised in Sweden, where climate and very high wages had been mainly responsible for the extensive research in this field. A careful study of site organisation and conclusions drawn from long experience was incorporated in present practice.

All the methods were directed towards two common aims: firstly, to reduce cost of labour and shuttering and secondly, rapid construction by means of modern equipment. A weakness of precast concrete, of course, was in the joints, and wet joints were usually provided. Because of heavy wind loads nominal reinforcement was provided, not only on vertical joints but also on the horizontal.

7. A recent trend in the search for economy, both in this country and abroad, was rather to compromise between precast and in-situ concrete. If a design was prepared in such a way as to incorporate simple units, then unit-type shutters which could be re-used and mechanically handled would provide an alternative economical means of construction.

8. Two principal methods had been developed for the construction of precast concrete:

(1) precast concrete in small prefabricated units weighing up to .7 of a ton, and

(2) construction of larger units up to 10 tons in weight, usually precast on site, or at gravel pits provided these were no more than 20 miles from site.

In some countries, especially in Sweden, a present trend was to eliminate the use of beams and columns in favour of a panelling system of slabs and walls.

Mr. Bishop, replying to the discussion, first endeavoured to clear up what appeared to be a misunderstanding. It was not the authors' intention, he said, to criticise designers; they had hoped the paper would provoke a discussion in which contractors would state their views in an assembly of designers. They had tried to put before the meeting the important factors—continuity of work, simplicity, and clarity of the structural concept—which affect the costs and charges of mechanised operations. They were not proposing that designers did not know their job, he was sure they did, but the intention of the paper was to ask, why not look at this other aspect of the work?

He then answered the various points of detail raised.

Utilisation of plant was a useful measure both of the essential simplicity of the construction and of the employment of plant. If, he said, the design of a building was such that operations followed each other in broken sequence so that the contractor had to do a bit of each at a time, there must be some slack in the programme much beyond what one imagined would be the normal time. If there were no slack there

would be a queueing problem. If, however, the system of construction was such that the contractor could deal completely with each operation in succession, organisation of work was much more simple. The authors had tried to bring before the meeting the concept of the utilisation factor. Generally, where there was a composite design the utilisation factor would be much lower than where there was a simple design. In fact only one set of observations had been taken with a crane fitted with recorders and these indicated a 50 per cent utilisation during the construction of a multi-storey building with a column and beam frame.

The example of construction quoted in the paper, using a hoist, was actual. It had a broken sequence of operations and, taking this into account, the contractor was doing quite well. The stair-well took a long time, partly because of the design of the stairs themselves, and in the alternative solution the flights of stairs were precast.

Expressing agreement with Mr. Bullen concerning the influence of holes in concrete on the cost of construction he said that one of his colleagues had studied this problem in detail and had found that the cost was proportional to the number of holes which in some jobs had a greater influence than the area of shuttering. In particular, on one apparently ordinary job where the contractor had estimated a reasonable price for concreting, details such as holes had caused the actual cost to be £16 instead of an estimated £6 per cu. yd.

With regard to the example shown from Sweden it was chosen because the originator had made a conscious attempt to provide a continuity of work for all gangs by separating fabrication and erection and had also simplified the services and finishings.

Incidentally, Mr. Bishop continued, in Sweden the very large variety of special types of buildings was very invigorating. There they seemed to be able to build quite successfully very many novel structural designs. But whereas the rate of wages in England was nominally 4s. per hour, the rate of wages in Sweden was, roughly, 12s. per hour. Thus savings in cost of labour and speed of erection could offset the cost of plant. If 12s. per hour was paid for wages or, as in America, 30s. per hour, more plant could be employed because the possible savings in man-hours were more important.

Summing up comments made in the rest of the discussion, he said that perhaps in one way the authors had tended to advocate building in large units. The examples selected attempted, more or less successfully to provide the conditions which made large scale mechanisation possible. Fabrication and erection were separated and each gang was given a steady run of work. Moreover each system embodied in fabrication some of the work of the finishing trades. And, as an incidental advantage these methods imposed a very strict discipline on both the designer and contractor ensuring that the functions of both were thought of as a whole. Whilst suggesting that designs based on large, loadbearing panels might prove fruitful the authors did not, however, imply that it was impossible to gain equal advantage with in-situ concrete.

Mr. Bishop continued, these systems of construction using large panels were in use in several different countries, apparently in open competition with traditional methods. On examination, however, there were special circumstances in each which justified and explained the use of highly mechanised systems of construction.

In America a very large number of standard warehouses and the like had been built cheaply of precast concrete units. The programme of construction was a consequence of a sustained demand by the armed services for standard buildings of simple design which naturally lent themselves to prefabricated construction. During the past few years a bewildering variety of designs had arisen in France. Their inception had been a consequence of a planned increase of threefold in the rate of construction in a few years with a resulting shortage of craftsmen and direct action by the Government to make it to the contractors' advantage to use mechanised methods of work. In Russia, where a considerable volume of work was carried out by these means, the long winter, a shortage of skilled labour, and a planned programme of building all led to mechanised systems of construction. In this country, Mr. Bishop said, although our dwellings were largely to a standard design and a continuous run of work was given by local authority housing, the relationship between the cost of labour and of plant and the relative cost of materials were such that a definite answer could not yet be given as to the probable economy of these methods in a free market here. This and the optimum rate of production were some of the questions which he and his colleagues were examining. A critical factor in such comparisons was the number of years over which the cost of plant was amortised and different choices could well reverse the result of any analysis.

Mr. Inglis had, quite properly, drawn the attention of the meeting to aesthetics and architectural considerations. The speaker believed, however, that in the past architecture had arisen directly out of technique and not vice versa. Therefore, if a structure represented an honest and not an artificial solution to a problem, the foundation should be laid for successful architecture.

There was a definite difference between an estimate prepared in a contractors' office for the purpose of a tender and estimates made to compare one system with another. If a contractor worked precisely from job to job he would have to have a detailed feedback of experience into his estimators' office, which very few had; contractors, he suggested, estimated within the limits of their general experience to yield an equal profit from year to year. If, however, one was making a definite comparison one had to go back to basic thinking, to proceed step by step and to consider the consequence of each step. He believed the best way to do that was by preparing a programme of work.

Mr. Bishop, in conclusion, stressed three points for consideration of designers.

Firstly there was a great deal of difference between forms of construction defining a broken sequence of operations and others which allowed each operation to be kept away from that following it. The authors considered the first step towards mechanisation was a reduction in the number of operations essential for construction rather than the development of complex machinery.

Secondly he urged designers not to seek economy in prefabrication by the elimination of a proportion of the work of a traditional trade. They might cut out two-thirds of the work but the saving in the cost of labour would be only a fraction of this. If the work of the traditional trades was to be eliminated by alternative methods the rule should be all or nothing.

Finally the use of plant would not work miracles. The effectiveness and economy of plant use was largely determined by the designer, on the drawing board. Given a suitable design mechanisation of building would yield a real saving in cost.

The President, on behalf of the meeting, thanked the lecturers for having engendered such an interesting discussion, and Mr. Bishop for the interesting way in which he had replied to it.

In a comment on Mr. Bullen's remarks on the provision of holes in structures for conduits and various other things, the President referred to the cost of holes in structures. In the pressure cabins of aeroplanes millions of pounds must have been spent on making holes to provide fleeting views for passengers.

Coming back to the lecture, he said the meeting had heard the rumblings of major new developments in the building of large office buildings and blocks of flats. He supposed the authors would hope that we were watching the beginnings of a mechanical engineering revolution in construction work, as we had experienced already in civil engineering. But he had a feeling that at any rate the older members and visitors, when thinking about the matter on their way home, would thank God for their country cottages!

Again he thanked the authors very much indeed for the manner in which they had dealt with their subject.

Institution Notices and Proceedings

SPECIAL JUBILEE ISSUE OF "THE STRUCTURAL ENGINEER"

To mark the Fiftieth Anniversary of the Institution, an extra number of "The Structural Engineer" will be published on the 21st July, 1958, and will be available to subscribers in all parts of the world, price 10s. 6d., on application to the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

One copy of this Special Jubilee Number will be issued free to every member of the Institution and also to all official Delegates to the Anniversary Conference to be held in October. Members may obtain additional copies at the published price on application to the Secretary.

The contents of the Jubilee Issue, which numbers 196 pages, fall into two parts, the first containing information concerning the Institution—its progress, ramifications and work. The second part, entitled "Fifty Years of Structural Engineering," comprises a series of technical articles by eminent members of the Institution. The President, Professor Sir Alfred Pugsley, O.B.E., F.R.S., contributes an introduction to the technical section and the subjects covered include Structural Engineering Research, Theoretical Analysis, Foundations and Sub-structures, Dock Structures, Dams and Retaining Walls, Concrete Making, Reinforced Concrete Structures, Design of Steel Frames for Buildings, Construction of Steel Framed Buildings, Bridges, Timber Construction, Aluminium Structures, Composite Construction and Clay Products and Brickwork.

FIFTIETH ANNIVERSARY CONFERENCE

Members of the Institution will have received particulars of the Fiftieth Anniversary Conference which is to be held in London on the 7th to 10th October, 1958. It would be greatly appreciated if those wishing to take part in the Conference would complete and return as soon as possible the Registration Form which was enclosed with the Programme.

Official delegates from technical bodies in many parts of the world are attending the Conference. This will commence on Tuesday evening, 7th October, with a Reception at Londonderry House, Park Lane, by Mr. G. S. McDonald, who will be installed as President of the Institution on the 2nd October, 1958. The Technical Sessions will be held on the mornings of Wednesday, Thursday and Friday, 8th-10th October. Tours of engineering interest will take place. Alternative tours and visits of general interest are available and the programme includes a variety of social functions and entertainments. The Conference will end with a Banquet and Dance at the Dorchester, Park Lane, on Friday, 10th October.

The technical Sessions will be held at the Institution's Headquarters, 11, Upper Belgrave Street, London, S.W.1., and also by courtesy of the Council of the Institution of Naval Architects in their premises in the adjoining building. The Lord Mayor of London, Sir Dennis Truscott, will give the Opening Address on Wednesday morning. Copies of all technical papers will be issued in advance to all registered participants in the Conference. In each session, all the papers will be briefly presented by one Reporter so as to allow the maximum possible time for discussion. Delegates and Conference members desiring to speak in the discussion are requested to submit their names before the meeting. At the end of each session the authors will be given the opportunity to reply briefly to the points raised. More detailed written replies may be incorporated in the Final Report to be published later. The technical papers cover a wide range of subjects and a list will be published in our next issue.

The Secretary of the Institution will be pleased to send additional Registration Forms to members on application. Others interested in the Conference are invited to apply to the Secretary at 11, Upper Belgrave Street, London, S.W.1., for copies of the Programme and Registration Form.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 22nd May, 1958, at 5.55 p.m. Professor Sir Alfred Pugsley, O.B.E., D.Sc.(Eng.), F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Gage, Roger Anthony, of North Harrow, Middlesex.
Glaves, John Joseph Thomas Hewitt, of Derby.
Howard, Douglas John, of Sidcup, Kent.
Matlab, Dhia Abdul Latif, of London.
Odiam, Alan Richard, of Johannesburg, South Africa.
Simpson, Alan Jeffrey, of Scunthorpe, Lincolnshire.
Willcocks, Anthony John, of Woking, Surrey.
Wilson, Kenneth Owen, of Wellington, New Zealand.

Graduates

Abreu, Benjamin Cyril, of Bombay, India.
Bhogal, Ajit Singh, of Mombasa, Kenya.
Binder, Alexander, of Bulawayo, Southern Rhodesia.
Chaturvedi, Narayana Rao Ramachandra, B.E. Mysore, of Bangalore, Mysore State, South India.
Cooper, Michael Lloyd, B.Sc. Nottingham, of Richmond, Surrey.
Duffell, James Roger, of Quorn, Loughborough, Leicestershire.
Gay, Peter, B.Sc. London, of Stanmore, Middlesex.
Goel, Harish, B.Sc. Agra, M.Sc. Agra, of Roorkee, U.P., India.
Hazzard, Anthony Oliver, B.Sc. Leeds, of Brentwood, Essex.
Husain, Syed Israr, of Jhelum, W. Pakistan.
Johnson, Roger Paul, B.A., M.A. Cambridge, of London.
Keymer, Keith Stuart, of London.
Lovegrove, Douglas John, of Victoria, B.C., Canada.
McDowell, Alan, of Workington, Cumberland.
Mahendran, Chinniah, of London.
Mehta, Hasmukh Shantilal, of Ahmedabad, India.
Muller, John Roman, B.Sc.(Civil) Rand, of London.
Ng, Lin-Hing, D.I.C., of London.
Okoye, Stephen Nwafor, of London.
Oliver, Jack Lewis William, of Orpington, Kent.
Raje, Arvind Ganpatrao, B.E.(Civil) Poona, of Kymore, India.
Tso Kai-Sum, B.Sc.(Eng.) Hong Kong, D.I.C., of London.
Wall, Denis Vincent, B.E.(N.U.I.), of Dublin, Eire.

Associate-Members

Allas, Leo, of Pitt Meadows, B.C., Canada.
Daniolos, Yani (Jean M.), of Mombasa, Kenya.
Ho Hon Chu, of Hong Kong.
Ismatullah, Chaudhry Mohd., of Sialkot, West Pakistan.
Komal, Mulkraj, of Banaras, India.
Miller, Michael Jenner, B.Sc.(Civil) Birmingham, of Scarborough.

Member

Connell, John William, of Melbourne, Victoria, Australia.

Transfers

Students to Graduates

Byrne, James Patrick, of South Harrow, Middlesex.
Choo Yut Shing, of Singapore.
Fozzard, John, of London.
Garrett, Maxwell John, of Brentwood, Essex.
Gonsal, Dugald Trystan Julius Herbert, of Colombo, Ceylon.
Jhaveri, Babubhai Chimanlal, of Bombay, India.
Lea, Norman Watson, of Mt. Wellington, Auckland, New Zealand.
Noble, Alfred Henry, of Johannesburg, South Africa.
Peskett, George John, of Thornton Heath, Surrey.

Graduates to Associate-Members

Abela, Albert, B.Sc.(Civil) Natal, of Durban, South Africa.
Bagnall, John Burfitt, of Chislehurst, Kent.
Billington, Roy, of Liverpool.
Biviji, Shiraz Abdeali, B.E.(Civil) Poona, D.I.C., of Willington, near Derby.
Elder, Brian Charles, of London.
Gould, Harold, of Kampala, Uganda.
Gould, Noel Brian, of Salisbury, Southern Rhodesia.
Higgins, Geoffrey Thomas, of Burgess Hill, Sussex.
Keyworth, Richard Stanley, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
Panesar, Ripdaman Singh, B.Sc.(Hons.) Wales, of Kampala, Uganda.
Poole, Clifford Jerrold, of Johannesburg, South Africa.
Ramarao, Bodapati Venkata, of Jabalpur (M.P.), India.
Rodel, George Wyndham Howard, B.Sc.(Civil Eng.) Natal, of Durban, South Africa.
Rumney, Derek Peter, of Stretford, Lancashire.
Sengupta, Asish, B.E. Calcutta, of Stockport, Cheshire.
Smethurst, Gordon James, of Bolton, Lancashire.
Tan Hang Seng, of Ipoh, Perak, Malaya.
Tantry, Prabhakar Vasudev, B.Sc. Bombay, B.E.(Civil) Poona, of Bombay, India.
Traverse, John Derek, of San Francisco, Calif., U.S.A.
Turabi, Dafalla Abdulla, B.Sc.(Eng.) London, of Liverpool.

Associate-Members to Members

Furtado, Leonard Joseph, B.E.(Civil) Calcutta, of Calcutta, India.
Hodgkinson, Allan, A.M.I.C.E., B.E.(Civil) Liverpool, M.E. Liverpool, of Virginia Water, Surrey.
Seddon, Albert Eric, B.Sc., M.Sc., Manchester, of Watford, Hertfordshire.
Whitaker, James Maurice, M.B.E., M.I.C.E., of London.

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, May 22nd, 1958, at 6 p.m. Professor Sir Alfred Pugsley, O.B.E., D.Sc.(Eng.), F.R.S., M.I.C.E., F.R.Ae.S., in the Chair.

The Secretary (Major R. F. Maitland, O.B.E.), read the notice convening the meeting.

The Minutes of the Annual General Meeting held on May 23rd, 1957, as published in *The Structural Engineer*, July, 1957, were taken as read and were confirmed and signed.

Mr. G. S. McDonald (Vice-President) moved the adoption of the Sessional Report of the Council and the accounts for the financial year, 1956.

Mr. L. E. Kent (Vice-President) seconded the motion, which was carried unanimously.

Mr. B. Scruby (Member of Council) proposed the re-election of Messrs. James Meston and Co., Chartered Accountants, as Auditors for the ensuing year. Mr. D. A. G. Reid (Member of Council) seconded the motion, which was carried unanimously.

The Secretary then read the report of the Scrutineers on the ballot for the election of President, the Honorary Officers and the Ordinary members of Council for the Session 1958-9, as follows :—

To : The Council of the Institution of Structural Engineers.

Gentlemen :

We, the undersigned, report that at the request of the President we have duly carried out the duties of Scrutineers of the Ballot for the election of Honorary Officers and Council for the Session 1958-1959, and we report accordingly as follows :

We received 1047 Ballot Papers, of which we rejected 28 as wholly spoiled and 22 as partly spoiled. We have attached a separate sheet showing the number of votes received by each candidate.

We declare the result of the Ballot to be as follows :—

Elected

PRESIDENT—Mr. G. S. McDonald, M.I.C.E., M.I.Mun.E.

VICE-PRESIDENTS—Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E., Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.C.E., Mr. F. R. Bullen, B.Sc.(Eng.), M.I.C.E., Dr. F. G. Thomas, Ph.D., B.Sc.(Eng.), M.I.C.E., Mr. E. N. Underwood, B.Sc.(Eng.), M.I.C.E., Mr. J. G. Hay, B.Sc.(Eng.), A.M.I.C.E.

HONORARY TREASURER—Mr. F. M. Bowen, M.I.C.E., Assoc.I.Mech.E.

HONORARY SECRETARY—Mr. J. Singleton-Green, M.Sc., M.I.C.E., A.M.I.Mech.E.

HONORARY LIBRARIAN—Mr. A. P. Mason, B.Sc., M.I.C.E.

HONORARY EDITOR—Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E.

HONORARY CURATOR—Mr. John Mason, B.A.(Cantab.), A.M.I.C.E.

The above are all elected for one year.

Elected as Ordinary Members of Council (London) Dr. A. R. Collins, D.Sc., M.B.E., M.I.C.E., Professor S. R. Sparkes, Ph.D., M.Sc., M.I.C.E., Mr. Kenneth Severn, M.C., M.A., M.I.C.E.

The above are all elected for three years.

Elected as Ordinary Member of Council (Country) Professor W. Merchant, M.A., S.M., D.Sc., A.M.I.C.E., A.M.I.Mech.E.

The above is elected for three years.

Elected as Associate-Member of Council (London) Mr. D. T. Williams.

The above is elected for three years.

Elected as Associate-Member of Council (Country) Professor P. B. Morice, B.Sc.(Hons.) (Eng.), Ph.D., A.M.I.C.E.

The above is elected for three years.

We are, Gentlemen,

Yours faithfully,

(*Signed*) C. B. BROWN
B. L. CLARK
R. D. MCMEEKIN
F. E. FOWLE
Scrutineers.

On a motion by the President, a vote of thanks was unanimously passed to the scrutineers.

EXAMINATIONS—JULY, 1958

The Examinations of the Institution will be held in the United Kingdom and overseas on Tuesday and Wednesday, July 15th and 16th, 1958 (Graduateship), and Thursday and Friday, July 17th and 18th, 1958 (Associate-Membership).

REPRESENTATION ON OUTSIDE BODIES

The Council have appointed the following Institution representatives :—

Manchester College of Science and Technology—Court of Governors

Dr. D. D. Matthews.

Union of Lancashire and Cheshire Institutes—Building Advisory Committee

Mr. W. Fitton.

City and Guilds of London College, Department of Technology—Advisory Committee on Fabrication of Steelwork

Mr. R. W. Schofield (vice Mr. D. A. G. Reid, resigned).

AWARD OF MERIT AND SCHOLARSHIP FUND

The Council has made an award of ten guineas each from the Award of Merit and Scholarship Fund to the following candidates in recognition of the high standard attained by them in the Institution's examinations for the Session 1956-57 :

Mr. F. Bell—Graduateship Examination.

Mr. P. T. Chad—Associate-Membership Examination.

LONDON GRADUATES' AND STUDENTS' SECTION

The following Honorary Officers and Committee members have been elected for the Session 1958-59 :—

Chairman : Mr. L. W. Debenham.

Vice-Chairman : Mr. A. S. Beeson.

Honorary Secretary : Mr. R. M. Amodia, B.E., 21, Wetherby Gardens, London, S.W.5.

Honorary Assistant Secretary (General) : Mr. B. C. Williams.

Honorary Assistant Secretary (Publicity) : Mr. A. E. Witchlow.

Honorary Treasurer : Mr. C. J. Rice.

EXAMINATIONS

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEGES

A Candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

PREPARATION FOR THE GRADUATESHIP EXAMINATION

Technical colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At Technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorites should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering. These cover only part of the requirements for the Associate-Membership Examination.

Colleges in List " A " provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

LIST " A "

Bath Technical College.

Belfast College of Technology.

Birmingham College of Technology.
Bolton Technical College.
Bradford Technical College
Bridgend Technical College.
Bristol College of Technology.
Chesterfield College of Technology.
Coatbridge Technical College, Lanarkshire.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal College of Science and Technology.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith College of Art and Building.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Portsmouth College of Technology.
Salford, Royal Technical College.
South-East London Technical College, Worsley Bridge Road, S.E.26.
South-West Essex Technical College, Walthamstow, E.17.
Southampton Technical College.
Stafford County Technical College.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List "B" provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List "B"

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton College of Advanced Technology, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle upon Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1958-59 :—

Chairman : Dr. D. D. Matthews.
Vice-Chairman : Mr. T. C. Waters.
Immediate Past Chairman : Mr. W. Fitton.
Honorary Secretaries : Mr. J. L. Robinson, 314, Northenden Road, Sale, Manchester ; Mr. M. D. Woods, 8, Dennison Road, Cheadle Hulme, Cheshire.
Honorary Auditors : Messrs. K. Norrey and G. S. Jones.
Committee : Messrs. W. Bates, H. J. Dowling, K. Norrey, F. C. Brookhouse, R. Gray, W. D. Blades, J. E. Guest, J. B. Storey, A. G. McNamara, J. B. Ashton, Professor W. Merchant, Professor J. A. L. Matheson.
Co-opted Member : Mr. A. S. Sinclair.

MIDLAND COUNTIES BRANCH

Hon. Secretary : J. R. Chaffer, M.I.Struct.E., 107, Jockey Road, Sutton Coldfield, Warwickshire.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : F. A. Butterworth, "Roscrea," Tansey Green, Pensnett, Brierley Hill, Staffs.

NORTHERN COUNTIES BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1958-59 :—

Chairman : Mr. D. W. Portus.
Vice-Chairman : Mr. W. R. Garrett.
Immediate Past Chairman : Mr. D. M. O'Herlihy, O.B.E.
Branch Honorary Secretary : Mr. H. W. Dowe, 2, The Crescent, Saltburn-by-the-Sea, Yorks.
Honorary Secretary (*Tyne Centre*) : Mr. J. Whitten.
Branch Honorary Treasurer : Mr. L. Dobson.
Tees Centre Committee : Messrs. A. V. Buttress, W. G. Gentry, O. Lithgow, F. S. Wilson, J. G. Henderson, R. Husband.
Tyne Centre Committee : Messrs. W. H. G. Durose, E. A. Parsons, L. Hodgens, A. Bone.

NORTHERN IRELAND BRANCH

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., A.M.I.C.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

Hon. Secretary : W. G. Cantlay, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.C.E., 3, Blairbeth Terrace, Burnside, Glasgow.

SOUTH WESTERN COUNTIES BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1958-59 :—

Chairman : Mr. E. W. Howells.
Vice-Chairman : Mr. A. N. Perkins.
Honorary Auditors : Messrs. F. W. Potter and F. J. Powell, M.B.E.
Honorary Treasurer : Mr. P. L. Harvey.
Honorary Secretary : Mr. C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.
Committee : Messrs. E. G. Cove, J. D. Norfolk, F. W. Potter, F. J. Powell, H. J. Scoles (*Past Chairman*), W. C. Tyler, L. F. Vanstone (*Past Chairman*).

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : K. J. Stewart, A.M.I.Struct.E., A.M.I.C.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

Hon. Secretary: E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Chairman: R. A. Sutcliffe, M.I.Struct.E.

Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Chairman: T. Karmakar, M.I.Struct.E.,

Hon. Secretary: W. N. Cursiter, B.Sc., A.M.I.Struct.E., c/o Redpath Brown and Co. Ltd., P.O. Box 648, Singapore.

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ADDITIONS TO THE LIBRARY

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TRA/280. *The Coefficient of Linear Expansion of High-Tensile Steel Prestressing Wire*, by P. R. James.

TRA/282. *A Crack and Condition Survey of 6 in. Unreinforced Concrete Roads in Housing Estates in the Borough of Worthing*, by R. I. T. Williams.

TRA/283. *A Comparison of the Compressive and Flexural Strengths of Concrete with and without Entrained Air*, by B. W. Shacklock and P. W. Keene.

TRA/286. *The Analysis of Linear Pin-jointed Frames on "Pegasus,"* by P. B. Morice. Library Translation No. 65. *Effect of Far-Advanced Compressive Strains of Concrete in Reinforced Concrete Beams Submitted to Bending Moments*, by Sven Sahlin.

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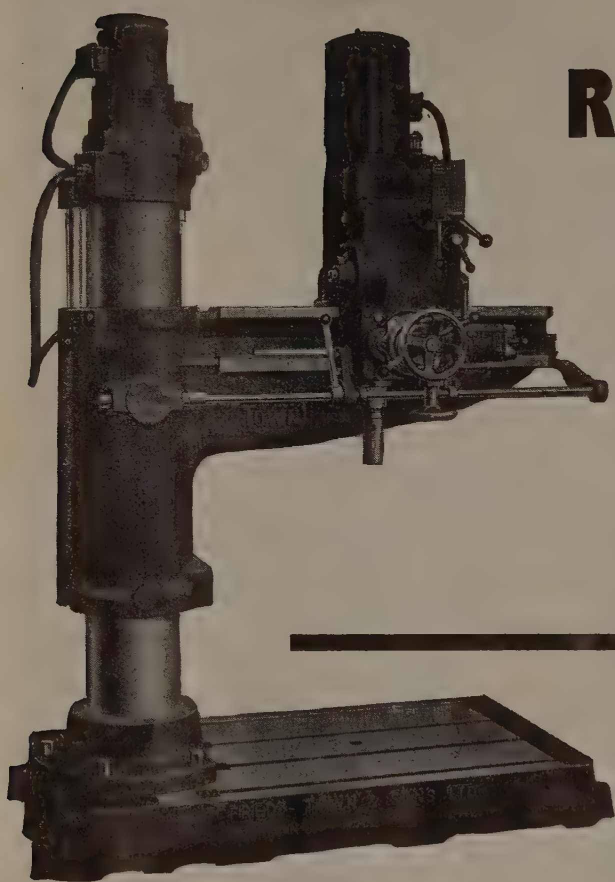
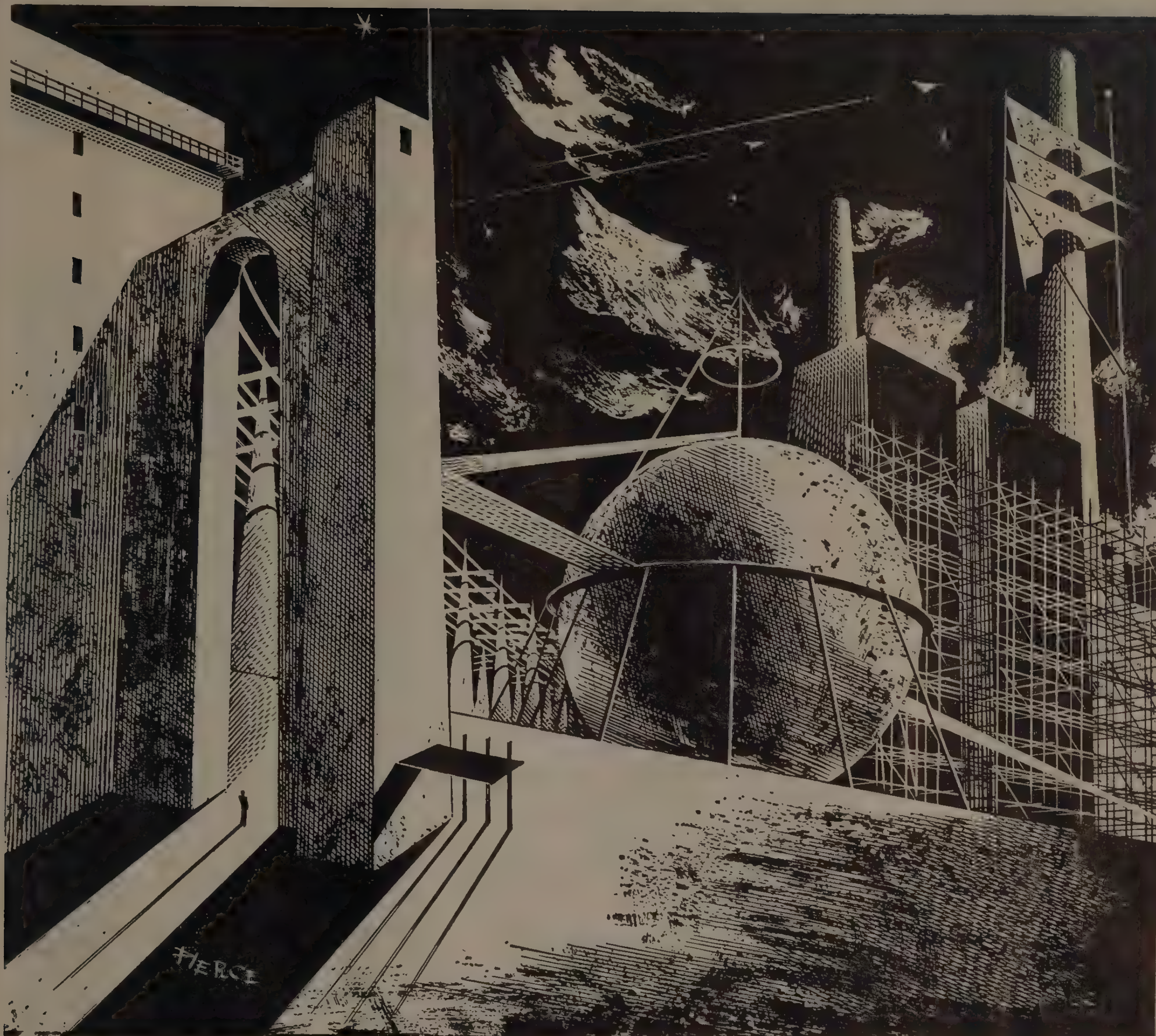
WANG, C. K. and ECKEL, C. L. *Elementary Theory of Structures*. New York and London, 1957. Presented by Mr. P. M. Grindrod.

YOUNG, J. McHardy. *Reinforced Concrete*, 3rd Edition. London, 1957.





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loads and unloads Tankers from 2,000 to 100,000 tons capacity

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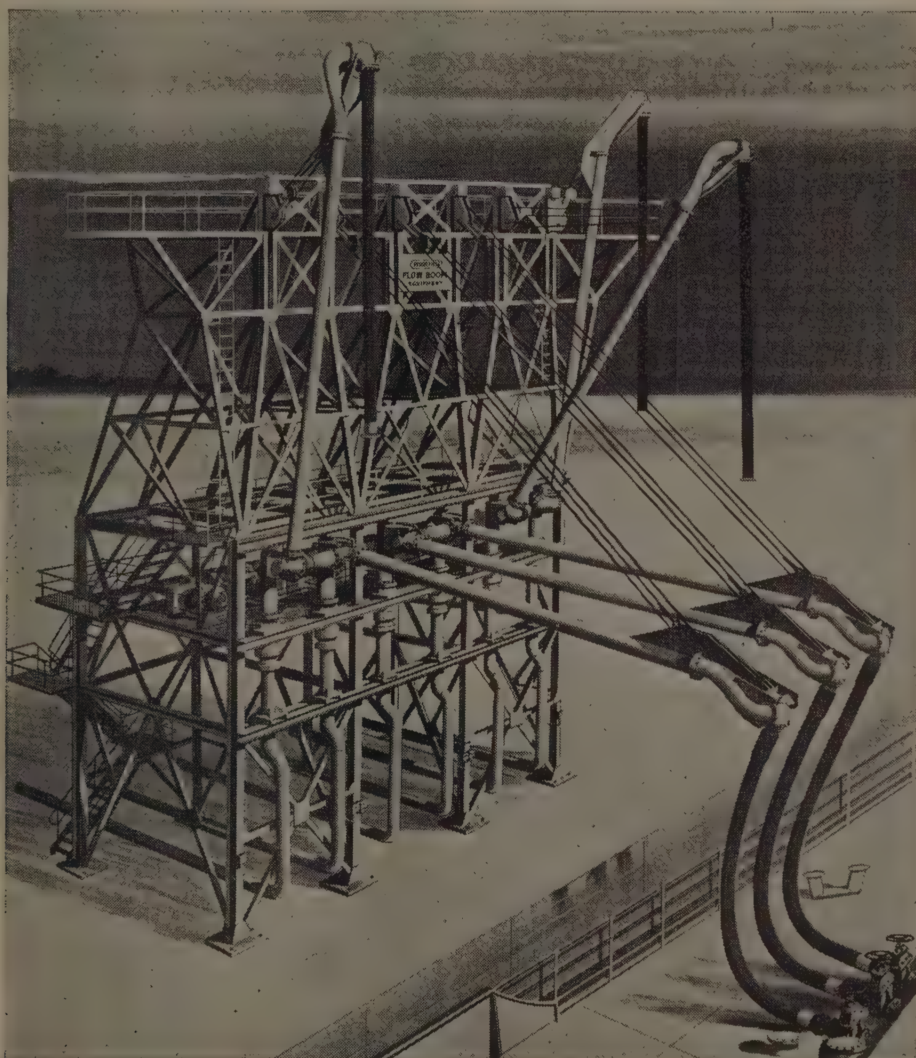

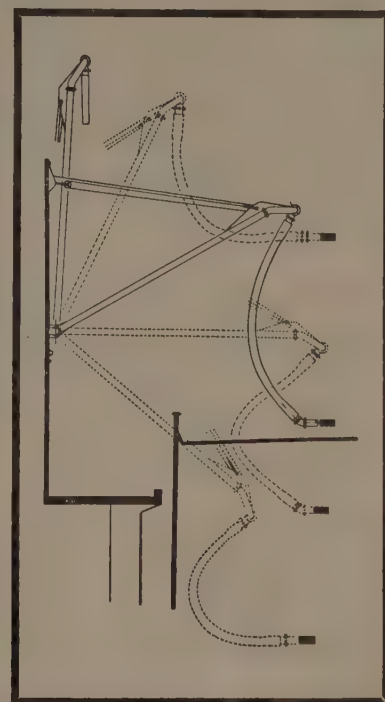

Diagram showing luffing motion of Flow Boom. This system can be designed to accommodate tankers ranging from 2,000 to 100,000 tons, and to suit local berthing conditions anywhere in the world.

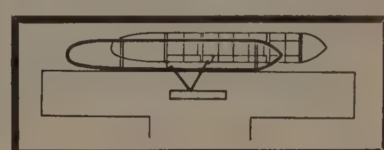

Diagram illustrating manner in which free slewing of Flow Boom permits safe tanker drift.

Woodfield Flow Boom Equipment features these vital advantages:—

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BUCKS County Council. Applications are invited for the appointment of a qualified Structural Engineer in the County Architect's Department on A.P.T. V £1,175 × £50(3) - £1,325 p.a. commencing salary according to qualifications and experience. A weekly allowance of 25s. 0d. and return fare home once every two months may be paid for six months to newly appointed married officers of the Council unable to find accommodation.—Applications, on forms provided, must be returned by 14th July, 1958. F. B. Pooley, County Architect, County Offices, Aylesbury.

AIR Ministry Works Design Branch requires in London, Structural Engineering Designer/Draughtsmen for reinforced concrete or structural steel work with sound technical training and several years varied experience in design/detailing of (a) Reinforced concrete construction for all types of buildings or (b) Steel framed sheds, warehouses and similar buildings. Salaries up to £1,015 p.a. Starting pay dependent on age, qualifications and experience. Long term possibilities with pensionable and promotion prospects. Five day week, 3 weeks 3 days leave a year. Normally natural born British subjects—Write stating age, qualifications, employment details including type of work done to any Employment Exchange quoting Order No. Borough 101.

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CONSULTING Engineers require qualified assistants. Senior men should be at least A.M.I.C.E. or A.M.I.Struct.E., with main experience in R.C. work, but versatile and able to design other materials and capable of handling the structure for large, and sometimes unusual buildings. Junior applicants must have potential capacity to develop to senior positions.—Write stating briefly qualifications and experience to Jenkins & Potter, 66, Victoria Street, London, S.W.1.

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REINFORCED Concrete Designers and Detailers with sound knowledge of framed structures are required for some interesting projects. The conditions are good and progress is assured for sound men who are able to accept responsibility.—Apply for an interview to E. J. Cook & Co. (Engineers) Limited, 54, South Side, London, S.W.4.

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